

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL

**INVESTIGATION OF THE PERFORMANCE OF DIFFERENT
TYPES OF REINFORCED CONCRETE STRUCTURES
UTILIZING NONLINEAR TIME HISTORY ANALYSIS**

ALPARSLAN DURMUŞ

**A THESIS OF MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING
PROGRAM**

ADVISOR: ASSIST. PROF. DR. ÜLGEN MERT

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LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

**FARKLI TİPTE BETONARME YAPILARIN ZAMAN TANIM
ALANINDA DOĞRUSAL OLMAYAN ANALİZ YÖNTEMİ
KULLANILARAK PERFORMANSLARININ ARAŞTIRILMASI**

ALPARSLAN DURMUŞ

YÜKSEK LİSANS TEZİ
İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI
DEPREM VE YAPI MÜHENDİSLİĞİ PROGRAMI

DANIŞMAN: DR. ÖĞR. ÜYESİ ÜLGEN MERT

HAZİRAN 2025



MASTER of SCIENCE JURY APPROVAL FORM

A thesis submitted by Alparslan DURMUŞ, defended on 02/07/2025 before the jury formed with the 30/06/2025 date and 2025/34 numbered decision of the GTU Graduate Administration Board, has been accepted as a MASTER of SCIENCE thesis in the Department of Civil Engineering, Earthquake and Structural Engineering Program.

JURY

MEMBER

(THESIS ADVISOR)

: Assist. Prof. Dr. Ülgen MERT

MEMBER

: Assoc. Prof. Dr. Onur ŞEKER

MEMBER

: Assist. Prof. Dr. Önder UMUT

APPROVAL

Gebze Technical University Graduate Administration Board

...../...../..... date and/..... numbered decision.

SIGNATURE/SEAL

ABSTRACT

Within the scope of this thesis, three different buildings designed according to the 2007 Turkish Building Earthquake Earthquake (TBEC 2007) were evaluated using a nonlinear time history analysis method in accordance with the 2018 Turkish Building Earthquake Code (TBEC 2018). Earthquake record was utilized based on the Kahramanmaraş Earthquake with a magnitude of 7.8, which occurred on February 6, 2023, in the Pazarcık district of Kahramanmaraş province. For nonlinear behavior model, fiber section models representing distributed plastic behavior were adopted in the plastic hinge regions of all columns and shear walls in all three buildings, while lumped plastic hinge models were used for the plastic hinge regions of beams. Based on these assumptions, the strain demands of the concrete and reinforcement fibers in the plastic regions of column and wall elements were compared with the deformation capacities of the respective elements. Similarly, the plastic rotation demands of the beams were compared with their plastic rotation capacities to complete the evaluation related to nonlinear ductile behavior. The evaluation of brittle behavior was carried out by comparing the strength demands and strength capacities of all structural system elements that transfer earthquake loads. In addition, global structural behavior parameters such as base shear forces, inter-story drifts, and roof displacement were also assessed. Furthermore, the mechanisms and proportions by which the earthquake energy imposed on the building was dissipated within the structural elements and the structural system as a whole were investigated. In summary, this study examined how three different buildings with similar plan geometries but different vertical structural members responded to the same earthquake record.

Key Words: Nonlinear Time History Analysis, Pushover Curve, Plastic Hinge, Moment Curvature, Plastic Hinge, Fiber Hinge Model, Modal Analysis, Mode Shapes, Equivalent Lateral Force, Ductile Damage, Nonductile Damage.

ÖZET

Bu tez çalışması kapsamında, Deprem Bölgesinde Yapılacak Binalar Hakkında Yönetmelik 2007 esaslarına göre tasarlanmış 3 farklı bina, zaman tanım alanında doğrusal olmayan çözüm yöntemi ile Türkiye Bina Deprem Yönetmeliği 2018 esaslarına göre değerlendirilmiştir. Deprem kayıtları için 6 Şubat 2023 tarihinde Kahramanmaraş İli, Pazarcık İlçesinde 7.8 büyüklüğünde gerçekleşen Kahramanmaraş Depremi esas alınmıştır. Doğrusal olmayan davranış modeli için her üç binada da tüm kolon ve perdelerin plastik şekil değiştirme bölgelerinde yayılı plastik davranışa karşılık gelen kesit hücresi lif modeli kabulü yapılırken, kirişlerin plastik şekil değiştirme bölgelerinde ise yığılı plastik davranış modeline karşılık gelen plastik mafsallık kabulü yapılmıştır. Bu kabullere göre kolon ve perde elemanlarının plastik şekil değiştirme bölgelerindeki beton ve donatı liflerinin birim şekil değiştirme talepleri, ilgili elemanların şekil değiştirme kapasiteleri ile karşılaştırılmıştır. Kirişlerin plastik dönme kapasiteleri ise yine aynı elemanların plastik dönme kapasiteleri ile karşılaştırılarak doğrusal olmayan sünük davranışa ilişkin değerlendirme tamamlanmıştır. Gevrek davranışa ilişkin değerlendirme ise deprem yükü aktaran tüm taşıyıcı sistem elemanları için dayanım kapasiteleri ile dayanım taleplerini karşılaştırarak tamamlanmıştır. Bununla birlikte taban kesme kuvvetleri, katlar arası göreceli kat ötelemeleri ve tepe deplasmanı gibi yapıların global davranışlarına ilişkin sonuçlar da değerlendirilmiştir. Ayrıca yapısal elemanlarda ve yapısal sistemin bütününde binaya gelen deprem enerjisinin hangi mekanizmalarla ne oranda sönmüldüğü de irdelenmiştir. Özetle, bu çalışma sayesinde farklı düzey taşıyıcı yerleşimine sahip benzer plan geometrisindeki üç farklı yapının aynı deprem kaydı altında nasıl bir davranış gösterdiği incelenmiştir.

Anahtar Kelimeler: Doğrusal Olmayan Zaman Tanım Alanında Çözüm, İtme Eğrisi, Plastik Mafsallık, Moment Eğrilik, Kesit Hücresi Lif Modeli, Modal Analiz, Mod Şekilleri, Eşdeğer Deprem Yüğü.

ACKNOWLEDGEMENTS

In the planning and preparation of this study, I would like to extend my thanks to my advisor, Dr. Ülgen Mert, for her support, knowledge, and experience. I am grateful to my family, who have supported me throughout my journey to this day. I also owe thanks to my colleagues Msc Cengiz Dal, Msc Muhammet Alarçin, Msc Süleyman Fırat Şentürk, Msc Hüseyin Özer and Burak Er, who have generously offered their assistance whenever needed in my work.



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LIST OF SYMBOLS AND ABBREVIATIONS

V_t	: Base shear
E_x	: Earthquake load x direction
m_t	: Total mass of structure
S_{aR}	: Reduced design spectral acceleration
T_p^x	: Natural period of x direction
I	: Building importance factor
S_{DS}	: Short period design spectral acceleration coefficient
$[M]$	: Mass matrix of structural system
$[K]$	: Rigidity matrix of structural system
$[C]$	: Damping matrix of structural system
$\ddot{u}(t)$	: Time depended acceleration vector of structural system
$\dot{u}(t)$	: Time depended velocity vector of structural system
$u(t)$	: Time depended displacement vector of structural system
$u(t)$	: Time depended displacement vector of structural system
$P(t)$	: Time depended dynamic seismic load vector of structural system
SRSS	: Square root of sum of squares
CQC	: Complete quadratic combination
R	: Combined total response for SRSS method
i	: Initial number of modes considered in the analysis
j	: Any number of modes considered in the analysis
n	: Number of modes considered in the analysis
ρ_{ij}	: Correlation coefficient between modes i and j
ζ	: Damping ratio
ω_i	: Natural frequency of i. vibration mode
ω_j	: Natural frequency of j. vibration mode
r	: Frequency ratio
Δ	: Displacement
$d\Phi$	: Rotation value of a infinitesimal piece of an section
dx	: Infinitesimal piece length for moment curvature
θ_p	: Plastic rotation value
Φ_u	: Ultimate curvature
Φ_y	: Effective yield curvature
L_p	: Plastic hinge length
k	: Stiffness
F	: Force
μ	: Ductility ratio
ϵ_u	: Ultimate deformation ratio
$\Delta i(ave.)$	: Average drift value
$\Delta i(max)$	: Maximum drift value
$\Delta i(min)$	: Minimum drift value
η_{bi}	: Torsional irregularity coefficient
$\theta_p(CP)$	: Collapse prevention level plastic rotation value
db	: Rebar area
$\theta_p(LS)$	: Life safety level plastic rotation value
$\theta_p(IO)$	: Immediate occupancy level plastic rotation value
$\epsilon_c(CP)$	: Concrete fiber hinge collapse prevention deformation level

$\sqrt{wwe}$	: Mechanic rebar ratio of effective confinement rebar
$\varepsilon_c(LS)$	: Concrete fiber hinge life safety deformation level
$\varepsilon_c(IO)$	: Concrete fiber hinge immediate occupancy deformation level
$\varepsilon_s(CP)$	: Steel fiber hinge collapse prevention deformation level
ε_{su}	: Ultimate unit deformation value for rebar
$\varepsilon_s(LS)$	: Steel fiber hinge life safety deformation level
$\varepsilon_s(IO)$	: Steel fiber hinge immediate occupancy deformation level
V_e	: Sesimic design shear force
V_r	: Total shear strength
b_w	: Width of the section
d	: Effective depth of the section
f_{ck}	: Characteristic compressive strength for concrete
V_{cr}	: Shear strength for concrete
f_{ctd}	: Design tension strength for concrete
γ	: Unitless constant
N_d	: Design axial load
A_c	: Section area
V_w	: Shear strength for rebar
A_{sw}	: Total tie rebar area
f_{ywd}	: Design yield strength of shear reinforcement
s	: Stirrup or transverse reinforcement spacing
A_{ch}	: Shear wall sross section
g	: Gravity acceleration
E_I	: Input energy
E_E	: Elastic strain energy
E_K	: Kinetic energy
E_H	: Hysteretic energy
E_P	: Potential energy
$(V_s)_{30}$	: Average shear wave velocity in the top 30 meters
$(N_{60})_{30}$	: Average standart penetration test blow count in the top 30 meters
c_u	: Undrained shear strength
n	: Live load participation factor
BKS	: Building occupancy classification

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1. INTRODUCTION

1.1. Motivation of This Study

In order to gain a deeper understanding of the Earth's internal structure, various geological and geophysical studies have long been conducted. As a result of these studies, a model of the Earth's structure has emerged, supported by the data obtained from these investigations. According to this model, the first layer along the path toward the Earth's core is the lithosphere, which is approximately 70–100 km thick. All geological events, such as earthquakes, occur within this layer. The layer located between the lithosphere and the core, which is about 2,900 km thick, is called the mantle.

In the lower part of the lithosphere lies the asthenosphere, also known as the soft upper mantle layer. Due to convection currents in this region, the lithosphere experiences fragmentation and breaks into pieces. Each of these fragments is referred to as a tectonic plate. The reason for the formation of convection currents is the high thermal energy released by radioactive decay occurring in the core region. As these convection currents rise, they cause stress in the lithosphere, which eventually leads to the fracturing of weaker zones and the division of the lithosphere into plates. The boundaries where tectonic plates that form the Earth's crust exert stress on one another are regions where earthquakes are frequently observed. When the stress between plates reaches a critical level in other words, when the friction between them is overcome deep, shock-like tremors can occur, spreading over hundreds of kilometers. The fractures that form between these two plates are called faults.

Due to its geographical location, our country is situated between the Eurasian Plate to the north and the Arabian Plate to the south. A significant portion of our territory lies within the region known as the Anatolian Plate. The boundary between the Eurasian Plate and the Anatolian Plate is known as the North Anatolian Fault, while the boundary between the Anatolian Plate and the Arabian Plate is referred to as the East Anatolian Fault.

Due to the geological reasons mentioned above, earthquakes classified as severe are frequently experienced in Turkey. In this study, the nonlinear behavior of three different buildings is evaluated based on the results of parametric models developed using the records of the February 6, 2023 Kahramanmaraş earthquake, which is one of these significant seismic events. The main objective of this thesis is to assess whether the essential requirements expected from a structure and its structural elements such as adequate strength, sufficient ductility, and sufficient stiffness are provided.

1.2. Literature Research

In this section, torsional irregularity, moment curvature, reinforced concrete wall frame and reinforced concrete moment frame academic studies discussed.

Semih S. Tezcan and Cenk Alhan selected a typical model building to investigate the effects of torsion on the moment and shear values of vertical structural elements. In this study, the positions of the shear wall elements were altered, and the modified plan type was analyzed for buildings with different story numbers. Based on a limited number of parametric analyses, the study recommends increasing the lower bound of the torsional irregularity coefficient to $h = 1.4$, while reducing the upper bound to $h = 1.8$ [12].

Gamal Hussein, Nasr Eid, and Hussein Khaled investigated the torsional irregularity behavior of various irregular buildings using simplified analysis, examining the impact of different levels of irregularity on buildings designed for seismic loads. The study concluded that buildings with severe irregularities deformed more due to torsional behavior compared to those with regular or less irregular configurations [13].

Adarsha A and Rajeeva S V evaluated the effects of torsion in buildings by analyzing five types of structures with different peripheral areas, where the locations of shear walls were varied for each plan type. The study concluded that when shear walls are symmetrically placed in both directions, the performance of the structure under dynamic lateral loads such as earthquakes and wind is significantly higher compared to that of other irregular structures. [14].

Ahmad Sheikh Abdallah and Safwan Chahal conducted structural analyses for two different systems a moment-resisting frame and a shear wall-frame system on two buildings with 8 and 12 stories located in specific seismic zones in Lebanon. Global

behavior parameters such as seismic response, time period, and maximum inter-story drift were evaluated. The results of the study indicated that the shear wall-frame system was more advantageous in terms of cost and lateral load resistance [15].

Anas Abou Lteaf conducted parametric modeling for both moment-resisting frame systems and shear wall-frame systems to understand the differences in structural behavior between the two types of buildings. The results of the analyses, including displacements and vibration modes, were compared, and findings related to stiffness and ductility were evaluated. It was concluded that systems composed solely of frames exhibit higher plastic deformations due to large displacements, and the positive impact of placing symmetrically distributed shear walls with adequate strength on structural performance was highlighted [16].

Saeid Foroughi and S. Bahadır Yüksel defined various reinforced concrete beam models with different parameters to investigate nonlinear moment-curvature relationships. The analytically examined parameters were calculated based on the provisions of TBEC (2018), ACI 318 (2014), ASCE/SEI 41 (2017), Eurocode 2 (2004), and Eurocode 8 (2004, 2005), as well as moment-curvature relationships. The aim of the study was to compare the effective section stiffness coefficients obtained from the analyses with those prescribed for reinforced concrete elements in different design codes. The results showed that for reinforced concrete beams with a constant tensile reinforcement ratio and concrete strength, an increase in the compression reinforcement ratio leads to higher values of the effective stiffness coefficient. In beams with constant tensile and compression reinforcement ratios, increasing concrete strength also increases the effective stiffness coefficient. It was demonstrated that the compression reinforcement ratio significantly influences the maximum moment capacity, effective flexural stiffness, effective stiffness coefficient, and ductility of the sections in reinforced concrete beams [17].

Iftekhar Anam and Zebun N. Shoma investigated the nonlinear behavior of reinforced concrete structures under static and dynamic loads. This nonlinearity was defined as geometric and material nonlinearity. Moment-curvature analysis was employed for beams. For dynamic analyses, the El Centro earthquake (1940) in the United States and the Kobe earthquake (1995) in Japan were used. The responses and internal force effects obtained from the dynamic analyses were found to be similar, with only minor differences [18].

In their 2015 study, Hüseyin Kasap, Necati Mert, Ezgi Sevim, and Begüm Şeber aimed to examine and compare the distribution of loads acting on structural elements in frame systems and shear wall-frame systems. The study involved varying the dimensions of columns and the locations of shear walls within the structural system. The mode superposition method was used for both static and dynamic analyses of the models. The results indicated that shear wall-frame systems are more rigid compared to frame systems, and that the majority of story shear forces are resisted by the shear walls. Additionally, it was observed that in shear wall-frame systems, structural behavior leads to significantly lower internal force demands on the frame elements [19].

Chetan Jaiprakash Chitte conducted a comparative analysis to understand the impact of irregularities on the capacity and ductility of structures. Pushover analysis was performed on four buildings with regular plan geometries. The results revealed that irregularities can cause differences of up to 40% in the structural behavior of buildings [20].

Shen and Akbaş (1999), in their collaborative study, proposed an energy-based design approach by predicting energy input and hysteretic energy in low, medium, and high-rise steel frame systems. Additionally, the study suggested that using single-degree-of-freedom systems for predicting the energy input of multi-degree-of-freedom systems may not be accurate [21].

1.3. Scope and Content

Within the scope of this study, three different types of buildings were analyzed using a nonlinear time history analysis method. Earthquake records from the 2023 Kahramanmaraş earthquake were used for the time-domain analyses. The nonlinear analyses were conducted using the ETABS v21 software.

Chapter 1 presents the definition, scope, and content of the problem addressed in this study, along with a review of the relevant literature. Chapter 2 provides the theoretical background related to the concepts involved in the study, while Chapter 3 outlines the methodology and roadmap to be followed in the research. Chapter 4 evaluates the results of the analyses, and Chapter 5 offers an assessment and discussion of these results.

2. SEISMIC DESIGN FOR EARTHQUAKE EFFECT

In this section of the thesis, information will be provided about the following topics:

- Analysis methods in structural design
- Energy based design

2.1. Analysis Methods

The structural design approach has undergone significant changes from past to present, especially in recent years with the advancement of software and hardware technologies. The development of these technologies, along with the increased availability of experimental data compared to the past, provides designers with the ability to create more realistic models. In structural design, analysis methods are categorized under subheadings as linear and nonlinear static and dynamic analysis methods. In static analysis, loads are applied to the building either slowly or as stationary forces, and it is assumed that the load does not change with time. Concepts such as sudden acceleration, vibration, and damping are not considered as parameters in this method. The analysis is completed by establishing equilibrium equations for this loading condition. In contrast, in dynamic analysis, the load varies with time. This method is especially considered for systems subjected to time-dependent loads such as earthquakes, wind, machinery-induced vibrations, and sudden explosion forces. In this analysis approach, parameters related to the dynamic characteristics of the building, such as vibration and damping, are taken into account. Below are the characteristic differences between these two methods, outlined under specific headings.

Static analysis:

- Loads are constant or applied slowly enough
- Loads do not vary with time
- The analysis is simpler and can be completed in a shorter time
- Deformations and internal forces are obtained

Dynamic analysis:

- Loads vary with time, and sudden loadings are also considered

- Applicable for all types of loading that can be defined as dynamic loads
- The analysis is more complex compared to static analysis
- Parameters such as vibration, resonance, and damping are taken into account
- Similar to static analysis, deformations and internal forces can be obtained

In linear analysis, the force–displacement relationship in the structure and its elements is linear, whereas in nonlinear analysis, this relationship is not linear. Below are the characteristic differences between these two methods, outlined under specific headings.

Linear analysis:

- The stress–strain relationship of materials is linear
- System stiffness is assumed to be constant
- Support conditions and connections are considered ideal
- Calculations are less complex and are completed more quickly compared to nonlinear analysis
- Useful for preliminary design and initial sizing in structural design
- Does not fully reflect the actual behavior
- May produce results that deviate from actual behavior in cases where stresses and displacements reach high values

Nonlinear analysis:

- The stress–strain relationship of materials is nonlinear
- System stiffness is not constant; it decreases under cyclic loading
- Support conditions and connections are not ideal; deformations related to plastic behavior are especially considered in support regions.
- Calculations are more complex and take longer time to complete compared to linear analysis
- It allows for the examination of limit states such as load-bearing capacity and collapse mechanisms in structural design
- It reflects the actual behavior of the structure more accurately compared to linear analysis

2.1.1. Linear Analysis Methods

According to the 2018 Turkish Building Earthquake Code (TBEC), three different methods are considered under linear analysis for defining earthquake loads. These methods are listed below. [2].

- Equivalent Lateral Force Method (ELF)
- Mode Superposition Method
- Mode Combination Method

In the Equivalent Earthquake Load Method, dynamic earthquake loads acting on a building are converted into an equivalent static load and applied to the structure as a static force (**Figure 2.1**). All internal forces and displacements are determined based on this linear static load assumption. Although this method is considered the most basic approach for accounting for earthquake loads, it can still be used for buildings of certain heights and with specific levels of irregularity. According to the 2018 Turkish Building Earthquake Code (TBEC), this method is applied based on equation 2.1 [2].

$$VtE_x = mt \times SaR \times (Tp_x) \geq 0.04 \times mt \times I \times S_{DS} \times g \quad (2.1)$$

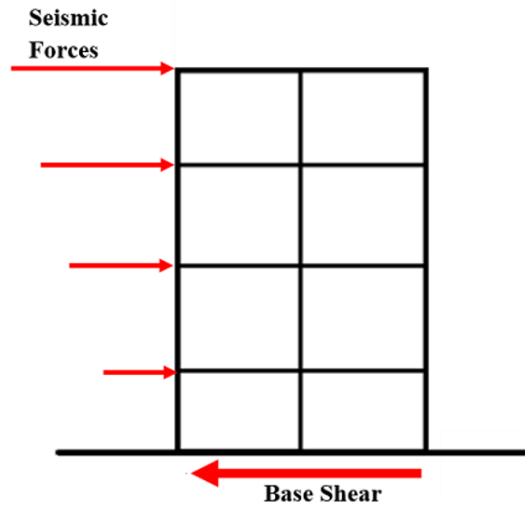


Figure 2.1: Equivalent lateral force method [2].

In the Mode Superposition Method, the building is analyzed separately for each vibration mode. To compute the mode shape vectors, an eigenvalue analysis is first performed to determine the eigenvectors of the structure. These eigenvectors represent the shapes that show how the structure vibrates in each mode. In general terms, these

mode shape vectors can be defined as vector quantities that describe how a structure vibrates in each individual mode. Regardless of how complex the dynamic response of a structure may seem, the actual behavior is essentially a combination of the contributions from each vibration mode of that structure. The equation used to obtain the mode shape vectors for each mode is referred to as the free vibration equation. For each mode, spectral acceleration, shear force, and displacement values are calculated and then combined using specific statistical methods to determine the total effect (**Figure 2.2**). These statistical methods are defined as SRSS (Square Root of the Sum of Squares) and CQC (Complete Quadratic Combination). The SRSS method is defined as the square root of the sum of the squares of any quantity obtained for each mode. While CQC can be used in all types of modal analysis, it provides more accurate results than SRSS when the vibration modes are closely spaced.

$$[M]\ddot{u}(t) + [K]u(t) = 0 \quad (2.2)$$

$$SRSS \rightarrow R = \sqrt{R_1^2 + R_2^2 + R_3^2 + \dots + R_n^2} \quad (2.3)$$

$$CQC \rightarrow R = \sqrt{\sum_i^n \sum_j^n R_i R_j \rho_{ij}} \quad (2.4)$$

$$CQC \rightarrow \rho_{ij} = \frac{8\zeta(1+r)r^{\frac{3}{2}}}{(1+r^2)^2 + 4\zeta^2 r(1+r^2)} \quad (2.5)$$

$$CQC \rightarrow r = \frac{\omega_j}{\omega_i} \quad (2.6)$$

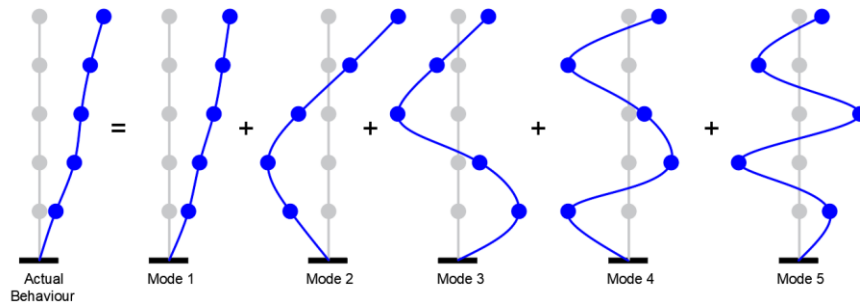


Figure 2.2: Modal superposition method physical interpretation [2].

In the Time History Modal Superposition Method, the dynamic equilibrium equation is directly solved based on actual or simulated earthquake records. For each mode of

the building, the time-dependent variation of typical modal response quantities such as displacement, inter-story drift, and internal force components is calculated. Since the response quantities of each mode are included simultaneously with their respective phase differences, there is no need to combine these results using statistical methods, as is required in the standard Mode Superposition Method.

2.1.2. Nonlinear Analysis Methods

The nonlinear analysis methods defined in TBEC 2018 (Turkish Building Earthquake Code) are listed below [2].

- Single-Mode Pushover Methods (Constant and Adaptive Single-Mode Pushover Methods)
- Multi-Mode Pushover Methods
- Nonlinear Time History Analysis Method

Pushover methods are generally classified as nonlinear static analysis methods. In these methods, earthquake loads are applied to the structure as equivalent static loads. The load–displacement relationship in the structural system is nonlinear. After defining the inelastic deformation behavior in the model, displacements corresponding to increasing earthquake loads are calculated. Once the target performance point is reached, deformation demands related to ductile behavior and internal force demands corresponding to brittle behavior are checked (**Figure 2.3**). In the Conventional (Constant) Single-Mode Pushover Method, earthquake load increments applied to the stories in each pushover step are defined in proportion to a fixed mode shape determined in the first step after applying non-seismic loads and kept unchanged throughout the pushover analysis. In the Adaptive Single-Mode Pushover Method, earthquake load increments applied to the stories and the corresponding displacement increments are defined proportionally to an updated mode shape. This updated shape is obtained from a new free vibration analysis considering the plastic hinges that have formed in each step after applying non-seismic loads. Multi-Mode Pushover Analysis is a more advanced version of the single-mode pushover method. It is used particularly for irregular, tall buildings or those with significant torsional irregularities where higher mode effects are important in understanding actual behavior. In this method, the eigenvalue problem is first solved to obtain mode shape vectors and periods for each mode. Then, separate pushover analyses are conducted for each mode, and

demands are determined. These results are then combined using certain statistical methods to derive the final outcomes.

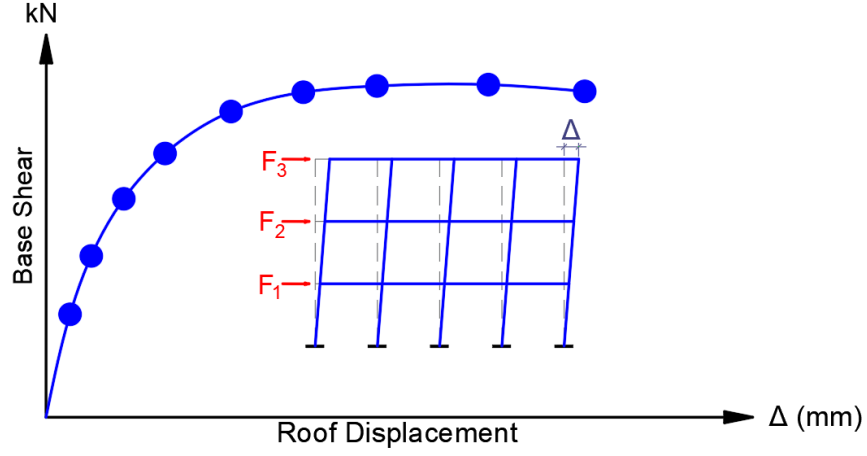


Figure 2.3: Ideal pushover curve [2].

Nonlinear time history analysis corresponds to the step-by-step direct integration of the set of differential equations that represent the equations of motion of the structural system under earthquake ground motion, using time increments. During this process, the time-dependent variation of the system stiffness matrix is considered due to nonlinear behavior. The equation of motion for the structural system, in its most general form, is given by equation 2.7.

$$[M]\ddot{u}(t) + [C]\dot{u}(t) + [K]u(t) = P(t) \quad (2.7)$$

In this equation, $[M]\ddot{u}(t)$ represents the inertial forces, $[C]\dot{u}(t)$ represents the damping forces and $[K]u(t)$ represents the stiffness forces, while $P(t)$ denotes the external load acting on the system. Mass, damping, and stiffness are defined in matrix form, whereas acceleration, velocity, and displacement are vector quantities.

2.2. Nonlinearity Types

Types of nonlinear behavior in structural systems are defined under the following categories. When a structure is subjected to strong ground motion, such as during an earthquake, it exhibits nonlinear behavior that incorporates all three types of nonlinearity listed below. Under strong ground motion, these three types of nonlinearity are key parameters that govern the nonlinear response of the structure.

- Geometric Nonlinearity
- Material Nonlinearity
- Boundary Conditions

Geometric nonlinearity refers to conditions in which displacements and rotations within the system cause significant changes relative to the system's original equilibrium configuration. This condition is often neglected in structural analysis and design; however, its impact becomes significant in structures with slender members, stiffness issues, or in tall buildings. In analyses where geometric nonlinearity is not considered, the stiffness matrix of the system remains constant. In contrast, when geometric nonlinearity is taken into account, the stiffness matrix varies at each step of the analysis. This is because, after deformations occur, the equilibrium equations must be re-established based on the deformed configuration of the system, meaning that the deformed structure is no longer geometrically identical to the original structure. In summary, the stiffness matrix becomes dependent on displacements and rotations. You can see **Figure 2.4** a typical example of geometric nonlinearity in structural elements is the P- Δ effect.

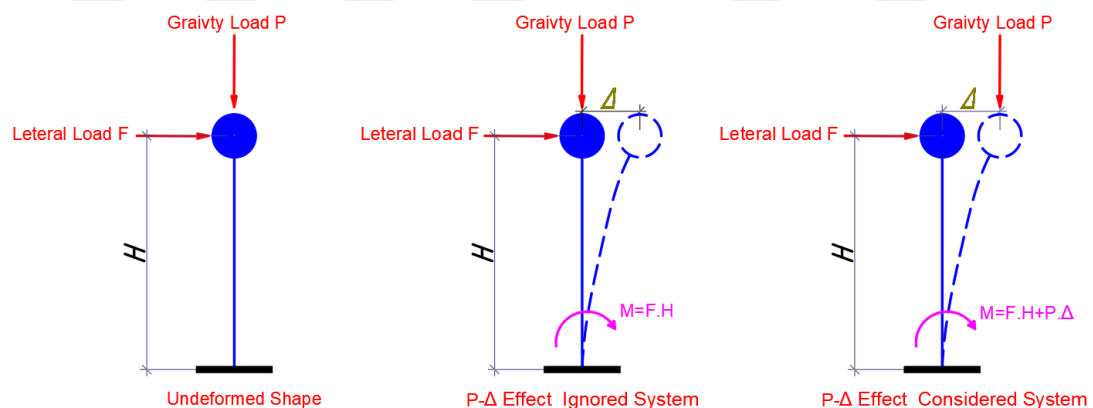


Figure 2.4: Typical P- Δ effect [2].

In material nonlinearity, Hooke's Law is no longer valid for the material's load-deformation graph. In other words, this relationship does not exhibit linear behavior. This typically occurs when ductile materials reach their yield point or when brittle materials experience cracking, crushing, or failure. In reinforced concrete structures, steel and concrete work together as a composite within the structural element. As is well known, concrete is a brittle material, while steel is ductile. When these two

materials act together, the resulting structural element is expected to exhibit a certain degree of ductile behavior. Therefore, the material behavior models and the manner in which reinforcement is placed within the cross-section have a significant impact on the overall ductile performance of the structure.

By nature, concrete is a heterogeneous material, and in all modern design codes, its tensile strength is assumed to be zero. On the other hand, concrete exhibits high strength under compressive loading. In Section 5.A.1 of the 2018 Turkish Building Earthquake Code (TBEC), a graphical representation (**Figure 2.5**) of confined and unconfined concrete behavior is provided below [2].

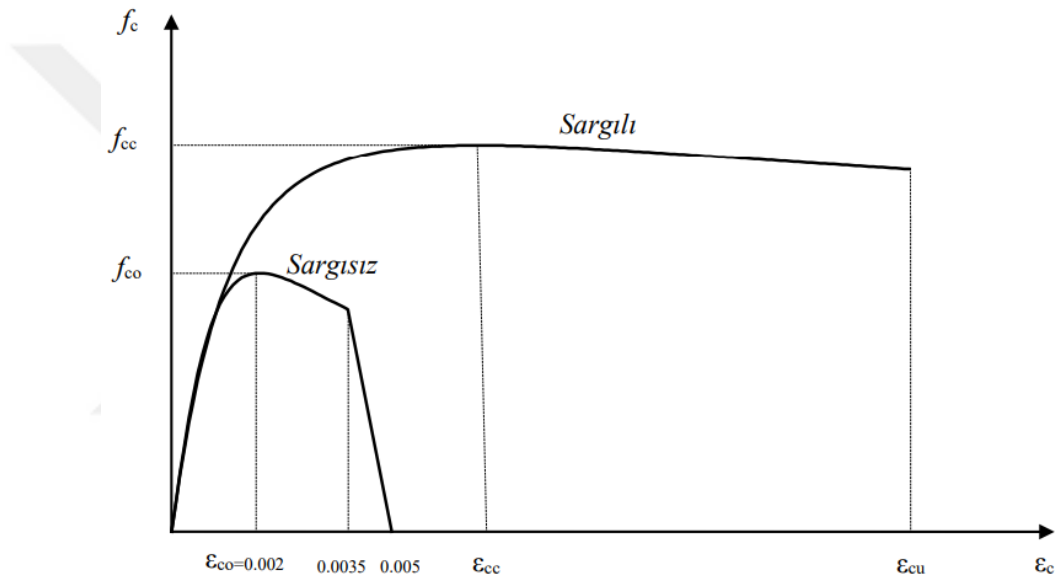


Figure 2.5: TBEC 2018 Confined and unconfined concrete model graph [2].

A visual representation of a reinforced concrete section containing both confined and unconfined concrete areas is shown in **Figure 2.5**. According to **Figure 2.5**, the area defined as the concrete cover in a reinforced concrete section is considered unconfined concrete, as it does not include transverse reinforcement. On the other hand, the core concrete area enclosed by the transverse reinforcement is defined as confined concrete.

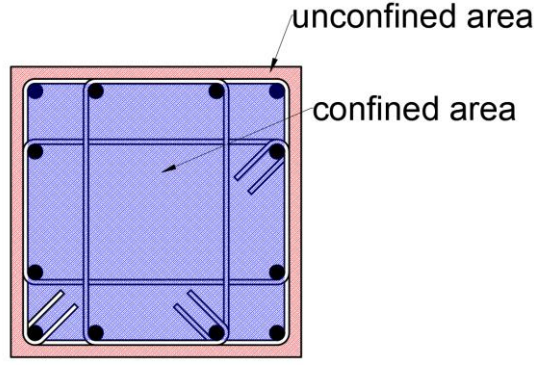


Figure 2.6: TBEC 2018 Confined and unconfined concrete areas [2].

Steel exhibits a behavior different from that of concrete. Unlike concrete, steel does not behave as a brittle material. It is a ductile material that responds similarly under both tensile and compressive forces. In Section 5.A.1 of the 2018 Turkish Building Earthquake Code (TBEC), the stress–strain curve of steel (**Figure 2.7**) and a table showing the strain limits for steel material classes (**Table 2.1**) used in reinforced concrete structures are provided [2].

Table 2.1: Rebar properties [2].

Kalite	f_{sy} (Mpa)	ϵ_{sy}	ϵ_{sh}	ϵ_{su}	f_{su} / f_{sy}
S220	220	0.0011	0.011	0.12	1.20
S420	420	0.0021	0.008	0.08	1.15 – 1.35
B420C	420	0.0021	0.008	0.08	1.15 – 1.35
B500C	500	0.0025	0.008	0.08	1.15 – 1.35

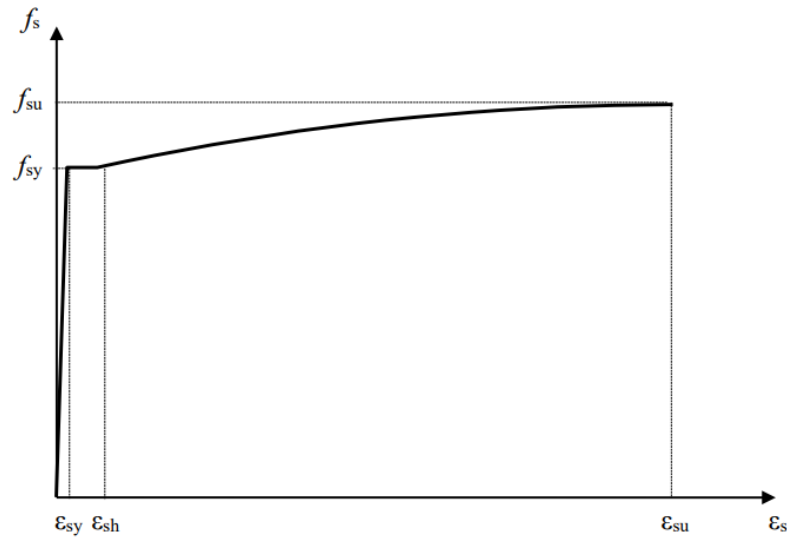


Figure 2.7: TBEC 2018 steel deformation limits table and graph [2].

Boundary nonlinearity arises when supports and connection points do not exhibit linear behavior. Theoretically, in a fixed (clamped) support, the rotation at the support point is zero, while in a simple support, the moment resulting from any deformation is zero at the end point. However, in actual behavior, a theoretically fixed support cannot possess infinite rigidity. Just as a hinge support can have a certain moment-carrying capacity, a fixed support also exhibits some degree of rotational flexibility. Under strong ground motion, supports may not respond linearly to applied loads and displacements. Springs and dampers that do not exhibit linear behavior are examples of this type of nonlinearity.

2.3. Hinge Definitions

According to the 2018 Turkish Building Earthquake Code (TBEC), two types of hinge definitions are prescribed for nonlinear analysis models: lumped plasticity and distributed plasticity behavior at the cross-sections [2].

According to the Lumped Plasticity Model, plastic deformations are concentrated in specific regions of finite length within the structural elements, and these regions are defined as the parts where the internal forces reach their plastic deformation capacities. This finite length is generally taken as half of the effective cross-section length of a structural element exhibiting flexural behavior in the relevant direction. It is assumed that all inelastic deformations occur within these finite regions, while the rest of the structural element behaves entirely elastically. The moment-curvature relationship is used to derive the input data for the lumped plasticity behavior. According to TBEC 2018, the lumped plasticity model can be used for all frame (bar) elements such as columns and beams, but it is not applicable to shell elements. **Figure 2.8** schematically illustrates the plastic hinge zones of a beam element subjected to simple bending.

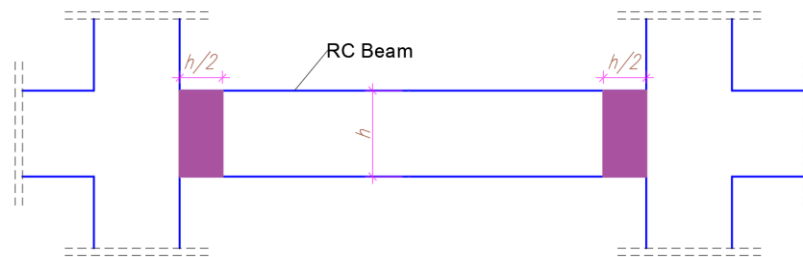


Figure 2.8: Plastic hinge zones for RC beam [2].

Curvature is defined as the rotation angle per unit length. For a beam element with an elastic curve, it is obtained by dividing the angle between any two cross-sections along the span of the beam by the length of the neutral axis between those sections. Rotation refers to the angle between these two deformed sections. In structural design calculations, the rotation value is expressed in radians, while the curvature value is given in rad/m. A simple cantilever beam, when deformed, will exhibit the type of deformation shown in **Figure 2.9**.

The Bernoulli-Navier Hypothesis is a fundamental assumption used in structural mechanics, especially for beams where shear deformations can be neglected. According to this hypothesis, cross-sections that are plane before deformation remain plane after deformation under bending behavior. When this hypothesis is considered in the analysis of a reinforced concrete beam, the normal stress distribution above and below the neutral axis of the cross-section is linear. This linearity simplifies the calculation of the section's capacity and the placement of reinforcement, thereby facilitating the design process for engineers.

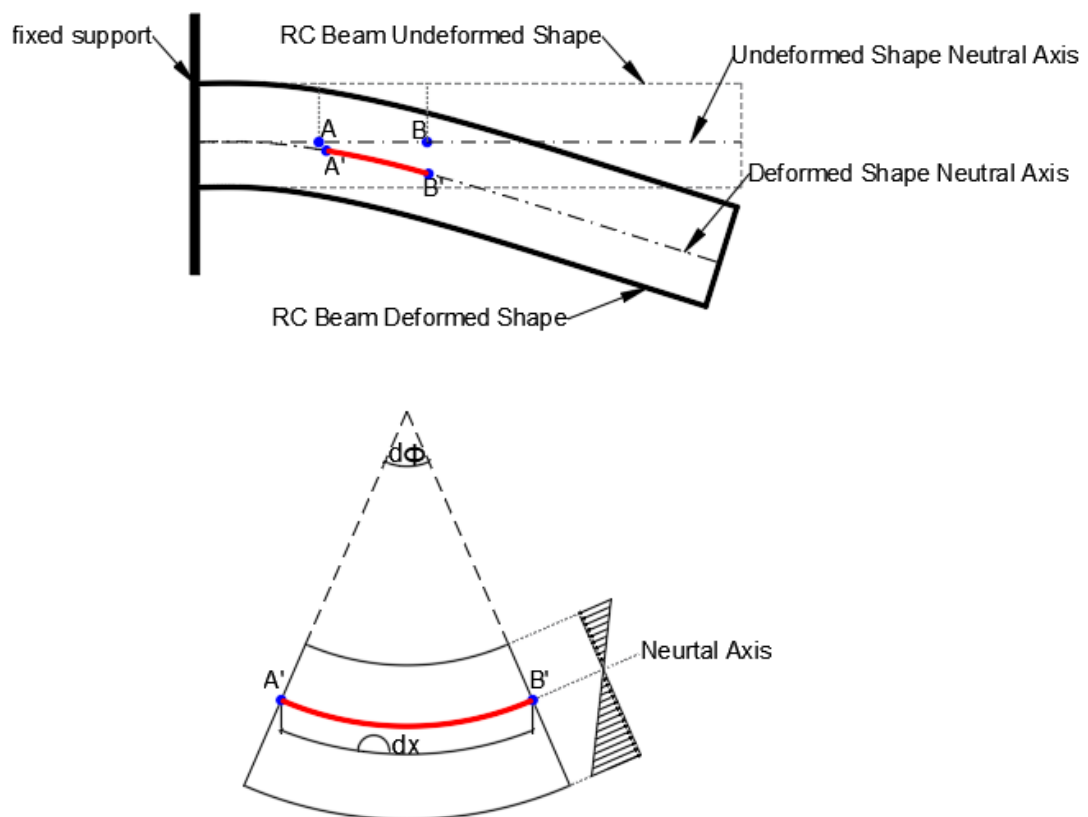


Figure 2.9: Typical RC beam deformed shape for curvature [2].

Here, $d\Phi$ represents the curvature angle of the relevant section. The curvature formula is given in equation 2.8.

$$Curvature = \frac{d\Phi}{dx} \quad (2.8)$$

Moment-curvature analysis is a type of analysis that provides significant insight into the structural behavior of a reinforced concrete section. In a ductilely designed beam subjected to pure bending, cracking first occurs in the concrete of the tension zone as the moment increases. With continued loading, the reinforcement in the tension zone reaches its yield strength, and the section continues to rotate with almost no increase in moment. When the concrete in the compression zone is crushed, the section reaches its collapse point. If a section subjected to earthquake loading attempts to undergo this behavior cyclically and is properly reinforced for ductility, the structural element will respond with high ductility capacity. In this way, the moment capacity of the element is preserved, ensuring adequate strength, while the earthquake energy imposed on the section is dissipated through inelastic deformations. **Figure 2.10** schematically illustrates the moment-curvature analysis of a reinforced concrete beam section. The results of this analysis can also be incorporated into calculations by applying appropriate linearization techniques.

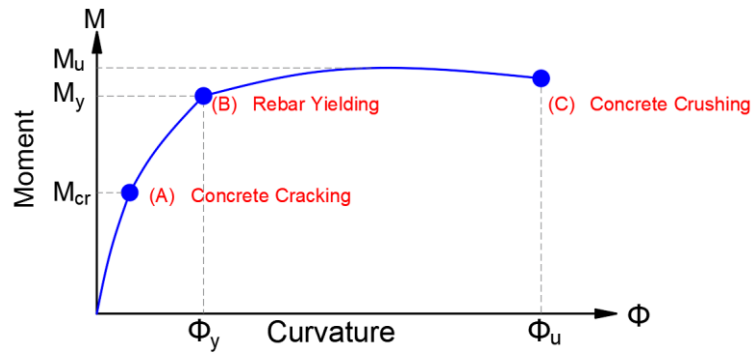


Figure 2.10: Typical moment-curvature graph for RC beam [2].

In nonlinear analysis of structural systems using the lumped plasticity model, the yield curvature and ultimate curvature values are important for evaluating the results. The yield curvature is theoretically considered the point at which nonlinear behavior begins. The ultimate curvature, on the other hand, corresponds to the point at which the section reaches its full rotation capacity. The rotational deformations occurring

between these two curvature values represent the plastic rotation capacity of the section, during which significant plastic deformations can occur without a substantial reduction in strength. The rotation occurring up to the yielding of the first reinforcement in the section is considered elastic rotation. When evaluating analysis results, considering the relationship between curvature and rotation, the plastic rotation value is defined as shown in equation 2.9.

$$\theta_p = (\Phi_u - \Phi_y) * L_p \quad (2.9)$$

In the distributed plasticity approach, the cross-section of a composite structural element—such as reinforced concrete, where steel and concrete act together—is defined using a fiber-based section model. Each of these individual components is referred to as a "fiber." The stress–strain relationships vary for each fiber type. For concrete fibers, the tensile components are typically not considered in the stress–strain diagram, whereas for reinforcement fibers, both tensile and compressive components are included. The fiber-based section model is used to more realistically represent the behavior of the cross-section under loading. According to the 2018 Turkish Building Earthquake Code (TBEC), there is no restriction on the use of fiber-based distributed plasticity models for columns, beams, and shear walls. [2]. **Figure 2.11** schematically illustrates the fiber elements used for the cross-sectional analysis of a column element.

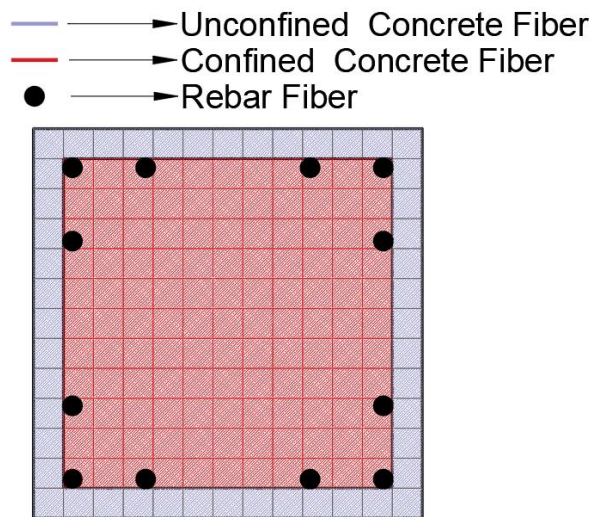


Figure 2.11: Typical fiber hinge mesh figure for RC section [2].

2.4. Frame, Frame+Shear Wall and Shear Wall Systems

According to the 2018 Turkish Building Earthquake Code (TBEC), cast-in-place reinforced concrete systems can be classified in terms of their ability to resist seismic loads as shear wall systems, frame systems, and shear wall-frame systems, where both walls and frames share the transfer of seismic loads [2]. These systems, as also outlined in the same code, are subject to certain limitations based on seismic design categories and building height classes. They can be designed as limited ductility, high ductility, or composite systems.

Frame systems are, in the most general sense, structural systems in which all earthquake loads are resisted by frames composed of columns and beams. Although these systems are widely used for resisting seismic forces, they generally have lower stiffness compared to shear wall or shear wall-frame systems, resulting in larger displacements. This necessitates designing frame systems with higher ductility. The main issues in frame systems include inter-story drift, second-order ($P-\Delta$) effects, shear safety of beam-column joints, and potential stability problems associated with large displacements. Despite having lower stiffness, frames possess a greater ability to undergo plastic deformations without losing strength from a nonlinear behavior perspective. If a frame system is properly designed to avoid brittle failure—meaning no structural elements are prone to brittle damage—then ductile behavior will manifest clearly in the system. The key reason frame systems can undergo inelastic deformations without losing strength is their high degree of redundancy and the presence of multiple potential plastic hinge regions. **Figure 2.12** schematically illustrates the possible plastic hinge locations in a frame system.

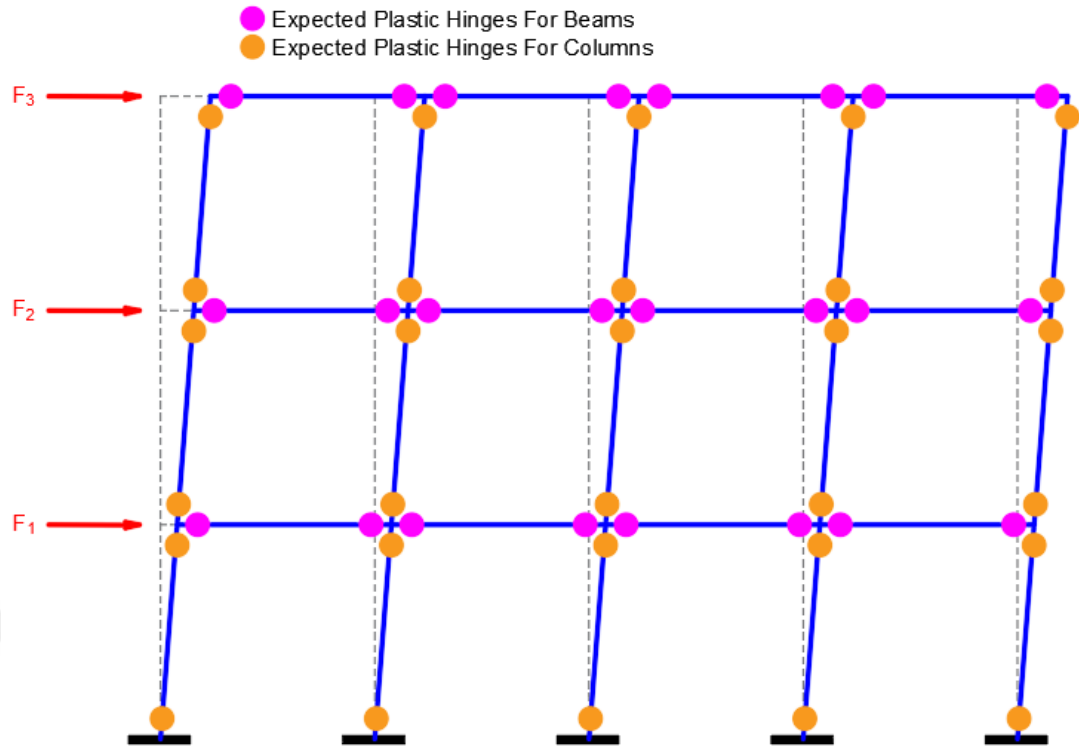


Figure 2.12: Expected plastic hinge locations for RC moment frame [2].

Shear wall-frame systems are generally used to limit the displacements of frame systems. As the number of stories in a building increases, displacements also tend to increase. If these displacements exceed permissible limits, brittle failures and second-order ($P-\Delta$) effects may occur. In such cases, shear walls play a role in increasing the stiffness of the system and mitigating these effects. When shear walls and frames are used together, the overturning moment caused by seismic loads is shared between the two systems—partially resisted by the walls and partially by the frames. The proportion of this moment shared between the walls and frames significantly affects the determination of the structural system behavior factor and the selection of ductility level. Compared to pure frame systems, shear wall-frame systems have a lower degree of redundancy. Unlike in frames, plastic hinges are not expected to form at every node in the walls. In shear walls, plastic hinges typically form at the base of the wall, which corresponds to the floor of the rigid basement level, and at floors where the cross-section of the wall changes significantly. Shear wall-frame systems are among the most commonly used structural systems in conventional reinforced concrete design. **Figure 2.13** schematically illustrates the potential plastic hinge locations in a shear wall-frame system.

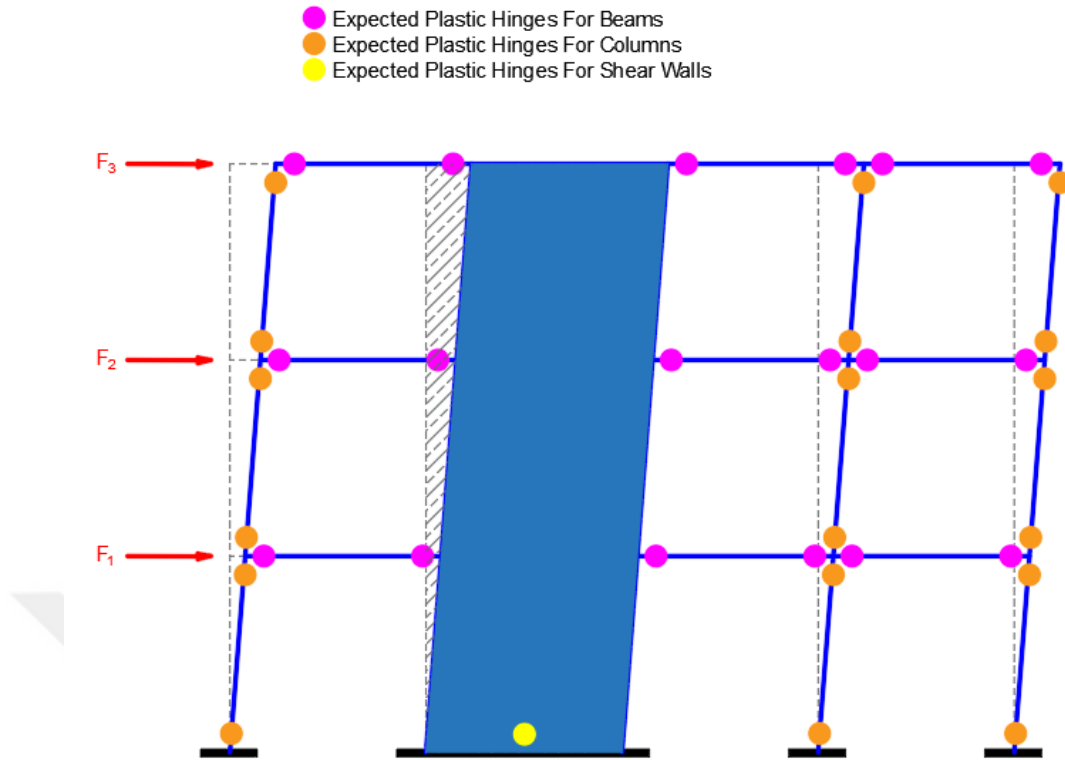


Figure 2.13: Expected plastic hinge locations for RC Wall- Frame System [2].

Shear wall systems are defined as buildings in which all seismic loads are resisted solely by shear walls, whether with or without openings. Compared to the other two systems, shear wall systems have the highest structural stiffness. In such systems, walls are often connected by beams, although it is also possible to design systems like tunnel form structures that include no beams at all. In these systems, columns are considered elements that carry only vertical loads, and their contribution to the overall stiffness of the system is minimal. While shear wall systems offer high lateral stiffness, they also have the lowest degree of redundancy. **Figure 2.14** schematically illustrates the potential plastic hinge locations in a shear wall system.

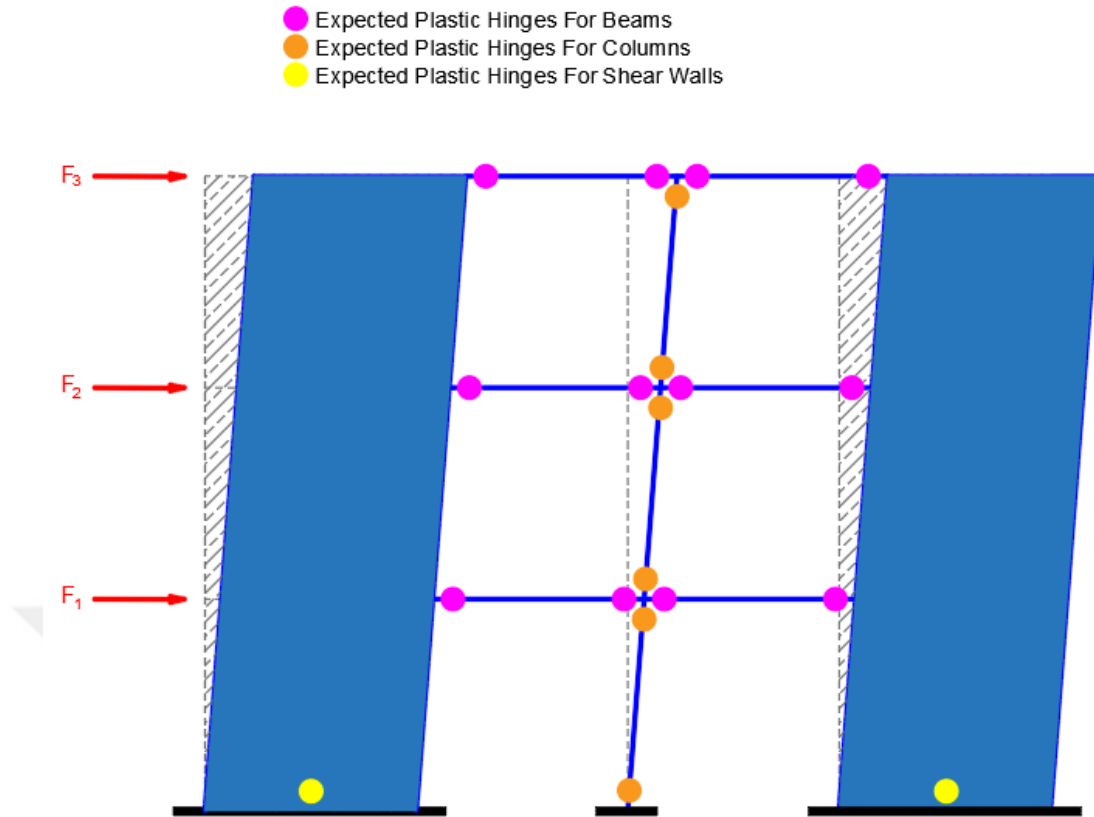


Figure 2.14: Expected plastic hinge locations for RC shear wall system [2].

2.5. Ductile and Brittle Failure Modes in RC

When a structural element is subjected to loading beyond the design expectations and reaches collapse not through sudden failure but by undergoing significant plastic deformation without a major loss in strength, this type of collapse is defined as ductile failure. Ductile failure is the desired mode of collapse in structural systems, as it possesses a high energy dissipation capacity under cyclic loading, such as that caused by earthquakes. Flexural failure is an example of ductile behavior. In the design process, it is expected that elements such as columns, beams, and shear walls exhibit classic flexural behavior leading to ductile failure. This type of failure is critically important for achieving the intended performance objectives of a structure.

Brittle failure refers to the sudden collapse of a structural element without prior warning when subjected to a load exceeding the design capacity. In reinforced concrete elements, failure modes such as shear, buckling, and punching are classified as brittle failures. Since this type of failure cannot dissipate energy under cyclic loading, it is

avoided in structural design. Nearly all modern seismic codes take precautions against such failure modes by introducing additional safety factors to reduce the likelihood of brittle failure.

2.6. Definitions of Stiffness, Ductility and Strength

Stiffness refers to the resistance of an element against deformation. In other words, stiffness is defined as the force required to produce a unit deformation in an element.

Figure 2.15 shows a visual representation of the load–displacement relationship for a simple cantilever column under lateral loading. Equation 2.9 describes the stiffness formula for the same column under lateral displacement.

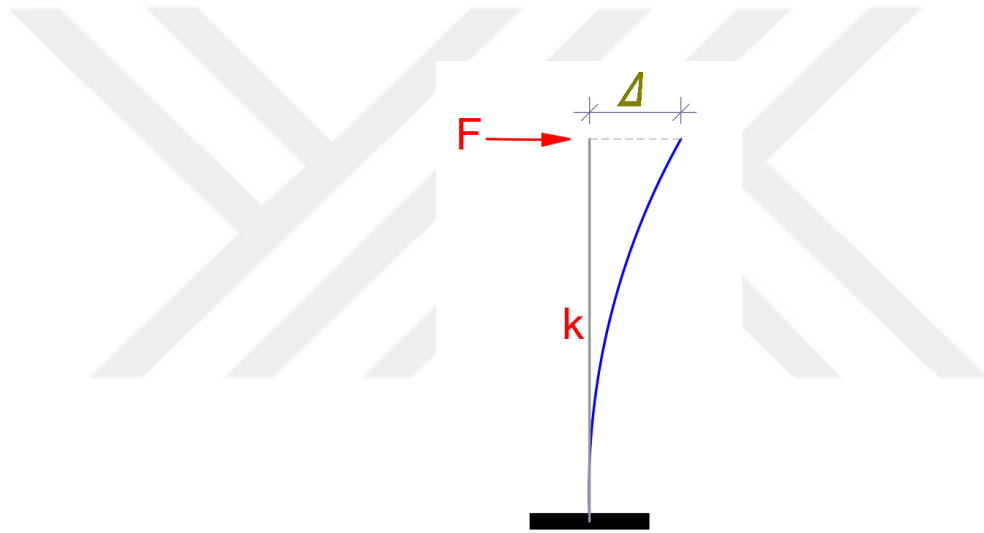


Figure 2.15: Sample column figure for rigidity definition [2].

$$k = \frac{F}{\Delta} \quad (2.10)$$

Ductility, in its most general definition, is the ability of a material to undergo deformation beyond the elastic limit without losing its strength. The higher the ratio of a material's ultimate strain to its yield strain, the more ductile the material is considered to be. **Figure 2.16** presents a graphical representation of ductility for a steel material exhibiting ideal elastoplastic behavior. Equation 2.10 provides the corresponding ductility formula for this material.

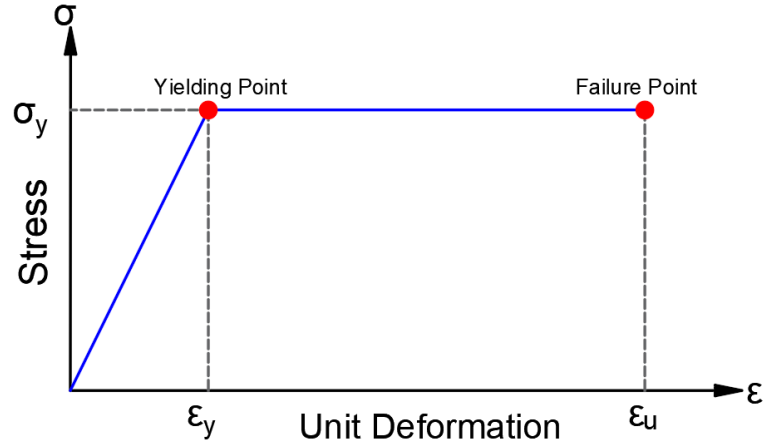


Figure 2.16: Sample stress and unit deformation graph figure for steel material [2].

$$\mu = \frac{\epsilon_u}{\epsilon_y} \quad (2.11)$$

Strength refers to the ultimate load-carrying capacity of a structural element under effects such as bending moment, torsional moment, axial force, and shear force.

All modern seismic design codes require a structure to have adequate strength, adequate stiffness, and adequate ductility. The absence of one or more of these three key characteristics means that the structure may not be able to achieve the expected performance under earthquake loading.

2.7. Torsional Irregularity Coefficient η_{bi}

According to the 2018 Turkish Building Earthquake Code (TBEC), the torsional irregularity coefficient is defined as the ratio of the maximum inter-story drift to the average inter-story drift on any given story for both earthquake directions [2]. This coefficient is denoted by the symbol η_{bi} and is expressed by equations 2.11 and 2.12. TBEC 2018 does not impose restrictions for buildings that exhibit this type of irregularity. However, if this value exceeds 2, it affects the determination of the earthquake analysis method. ASCE 7-22 uses the same calculation as TBEC 2018 for the torsional irregularity coefficient, but defines buildings with a coefficient greater than 1.4 as having "extreme torsional irregularity" and sets limits for seismic design categories for such buildings. For further details on torsional irregularity, ASCE 7-22 refers to FEMA P-2012. The torsional irregularity table from this standard is shown in the **Figure 2.17** and **Table 2.2** [4].

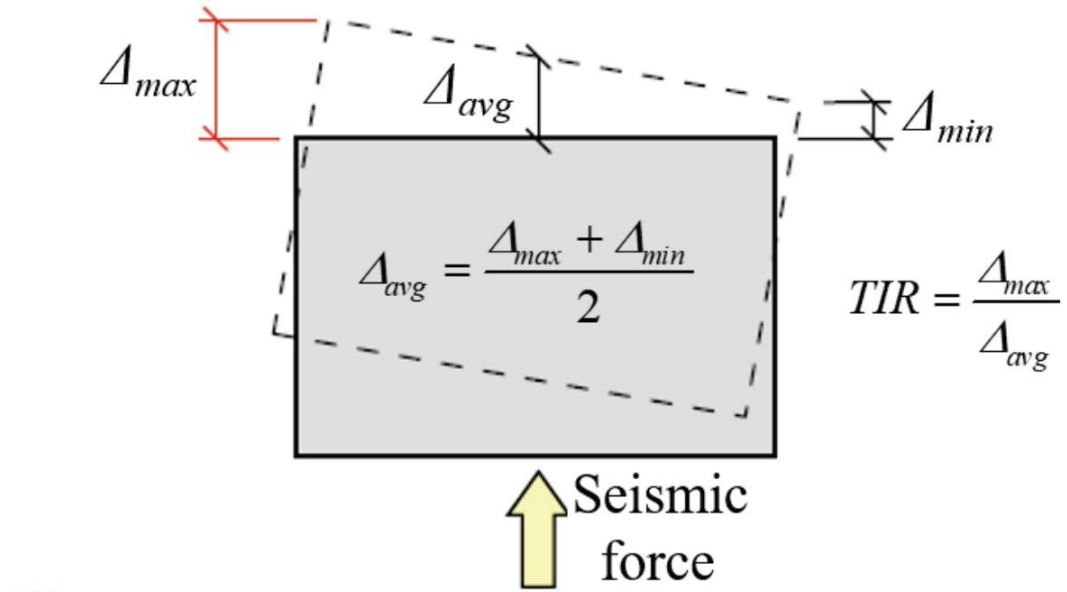


Figure 2.17: Torsional irregularity for a story [4].

$$\Delta i(ave.) = \frac{\Delta i(max) + \Delta i(min)}{2} \quad (2.12)$$

$$\eta_{bi} = \frac{\Delta i(max)}{\Delta i(ave.)} > 1.2 \quad (2.13)$$

Table 2.2: Torsion coefficient limits ASCE 7-22 [4].

Torsional Irregularity Classification	Triggering Criteria for the Irregularity	Add Requirements	Remove Requirements
Type 1a. Torsional irregularity	- $TIR > 1.2$ - Add additional trigger if >75% of strength on one side of the CM.	- Use 100%-30% orthogonal load combination rule	- N/A
Type 1b. Extreme torsional irregularity	- $TIR > 1.4$	- Use 100%-30% orthogonal load combination rule	- Automatic $\rho = 1.3$ - SDC E and F prohibition - ELF prohibition
Handle with code text (i.e., not a defined irregularity category)	If one or more of the following occurs: - $TIR > 1.4$ in both orthogonal directions - Lines of lateral resistance all on same side of CM	- Use 100%-30% orthogonal load combination rule - $\rho = 1.3$	- SDC E and F prohibition - ELF prohibition

2.8. Energy Based Design

Earthquakes introduce an energy input into structures. According to the law of conservation of energy, this input energy must be dissipated through specific mechanisms within the structural system. Accurately identifying how and through which mechanisms the earthquake energy is dissipated within the structure plays a crucial role in understanding the global structural behavior under cyclic loading.

When dividing the energy input, it can be categorized into two types: recoverable energy and non-recoverable energy. Recoverable energy consists of two components: elastic energy and kinetic energy. Non-recoverable energy, on the other hand, includes damping energy and inelastic energy.

Below is the equilibrium equation formulated for energy-based design. The terms in the equation represent, respectively: time-dependent input energy (EI), elastic strain energy (EE), kinetic energy (EK), hysteretic energy due to nonlinear behavior (EH) and damping energy (ED) [21].

$$E_I(t) = E_E(t) + E_K(t) + E_H(t) + E_D(t) \quad (2.14)$$

3. METHODOLOGY

3.1. Flow Chart

Within the scope of this thesis, three different buildings were analyzed using the Nonlinear Time History Analysis (NLTHA) method. The first structure is a system in which earthquake loads are resisted solely by high-ductility moment frames. The second structure combines high-ductility shear walls and high-ductility moment frames. The third structure also consists of high-ductility shear walls and frames, but incorporates torsional irregularity. Using nonlinear time history analysis, the global behavior of these three buildings—such as inter-story drift, roof displacement, and base shear force—was evaluated. Additionally, the deformation demands corresponding to ductile design and the strength demands corresponding to brittle behavior of the structural elements were checked to determine whether they exceed the permissible limits specified in TBEC 2018. The same cross-sections and reinforcement layouts were used for all frame elements across the selected structural systems. All reinforcement arrangements were chosen in accordance with the detailing requirements for high-ductility shear walls, columns, and beams as defined in TBEC 2018 [2]. For the alternatives that include shear walls, the reinforcement layout of the walls and coupling beams was kept identical, with the coupling beams designed according to high-ductility beam criteria. **Figure 3.1** shows flow chart within the scope of this thesis.

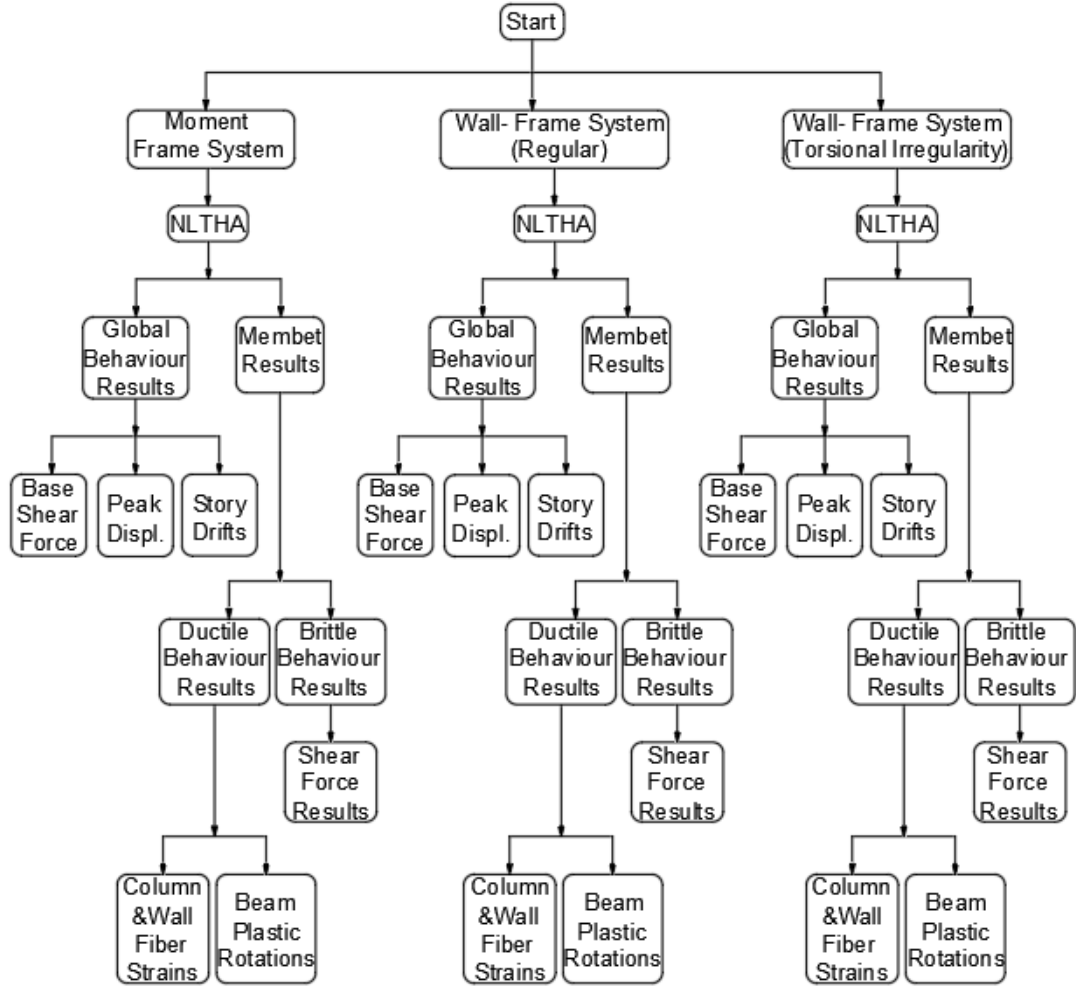


Figure 3.1: Estimated cases of flow chart utilizing NLTHA.

3.2. Selection of Earthquake Records

The selection and scaling of earthquake records for Nonlinear Time History Analysis (NLTHA) is a specialized area that requires expert knowledge. Within the scope of this thesis, earthquake data from the February 6, 2023 earthquake with a magnitude of 7.8, which occurred in the Pazarcık district of Kahramanmaraş Province, was used. The records obtained from the AFAD Gaziantep Islahiye station were utilized, and the earthquake data were evaluated for structural types assumed to be located at this station, with a local site class of ZC [8].

3.3. Evaluation Method of Analysis Results

Within the scope of this thesis, all three buildings were analyzed using the Nonlinear Time History Analysis Method in accordance with the 2018 Turkish Building Earthquake Code (TBEC), based on the earthquake records from the February 6, 2023 Kahramanmaraş Pazarcık earthquake [2]. Based on the analysis results, the following parameters related to the global behavior of the structural systems were evaluated.

- Base shear
- Joint displacement
- Column fiber hinge results
- Beam plastic hinge results
- Interstory Drift
- Shear wall fiber hinge results
- Shear wall shear force results

Similarly, for the structural elements of these three building types, the deformation demands corresponding to ductile behavior and the results related to brittle behavior were also evaluated. The evaluation criteria for structural elements are listed below.

- Column fiber hinge results (Ductile Behaviour)
- Beam plastic hinge results (Ductile Behaviour)
- Shear wall fiber hinge results (Ductile Behaviour)
- Shear wall shear force results (Brittle Behaviour)

The energy demand parameters listed below will be examined within the scope of the thesis:

- Energy compenents
- Energy compenents for groups

4. CASE STUDY OF NUMERICAL ANALYSIS

4.1. Input of Numerical Analysis

4.1.1. Analysis Models

The three different buildings assumed to be designed in accordance with TBEC 2018 each have six stories with a story height of 3 meters. Within each building, the typical floor plans are identical. The plan geometry, bay widths, and dimensions of columns and beams are the same across all three buildings. The only difference lies in the arrangement of the vertical load-bearing elements. The systems are named based on how they resist seismic loads, as listed below.

- Moment Frame System → Building A
- Wall- Frame System (Regular) → Building B
- Wall- Frame System (Irregular) → Building C

The plan, section and 3D of Building A is shown **Figure 4.1**, **Figure 4.2** and **Figure 4.3**.

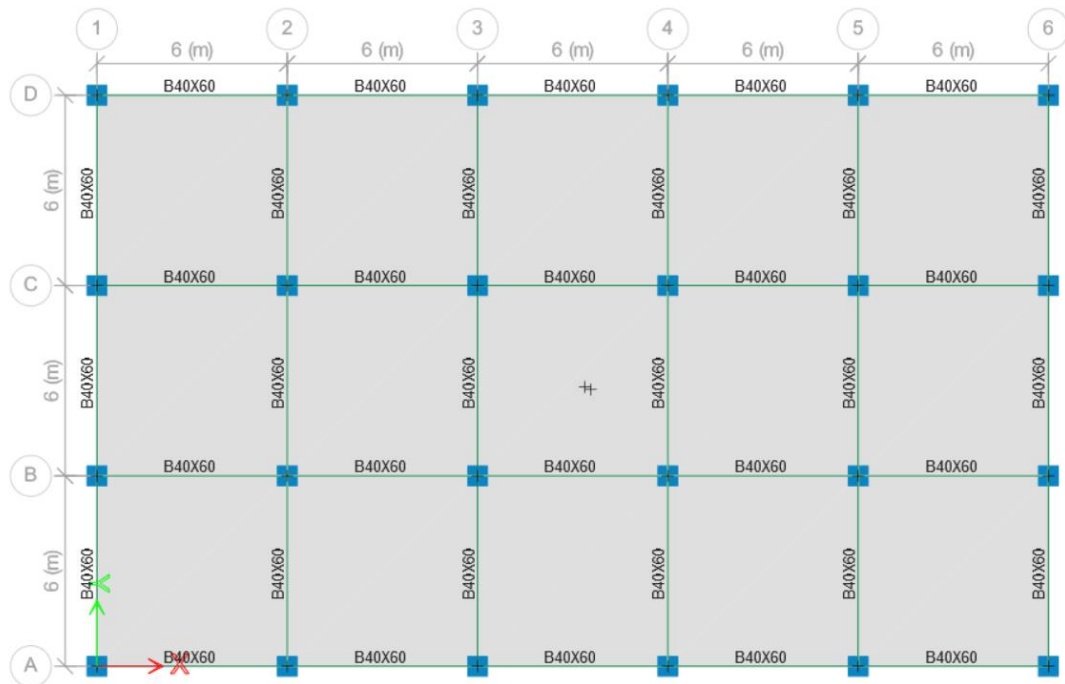
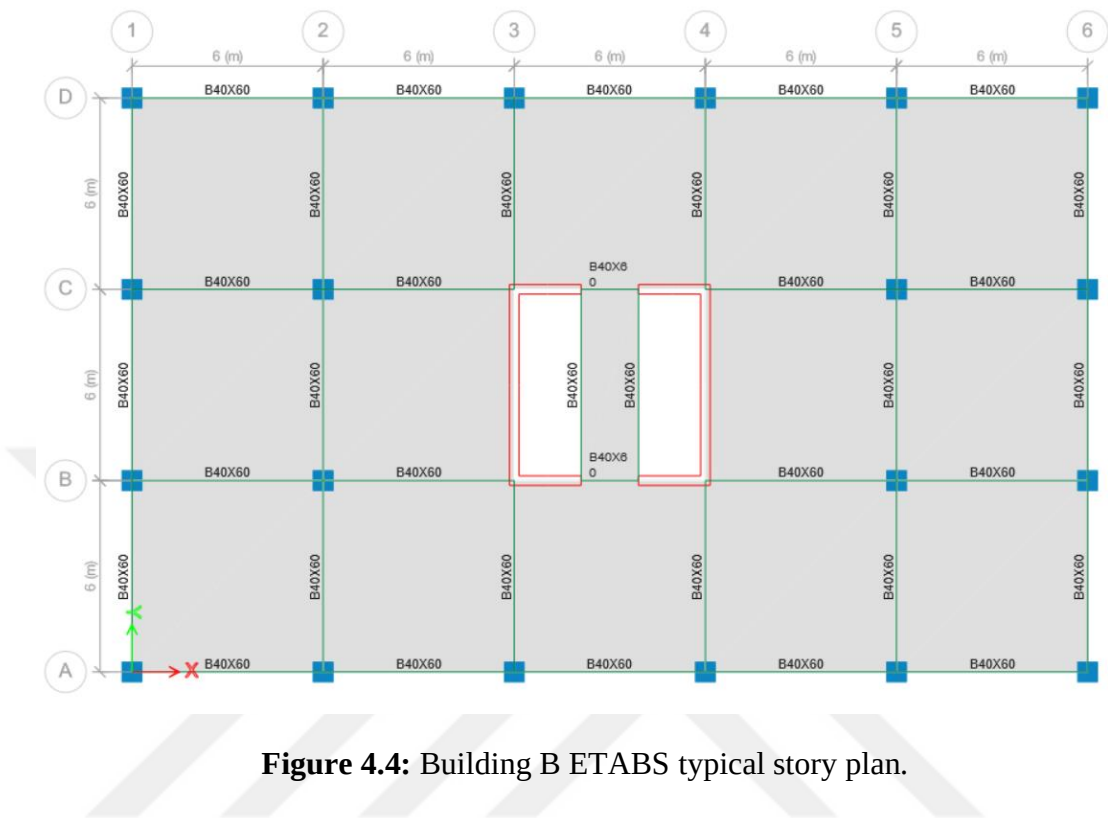


Figure 4.1: Building A ETABS typical story plan.



A 3D perspective view of a multi-story building frame structure. The structure consists of a grid of vertical columns and horizontal beams. The columns are colored blue, and the beams are colored green. The structure is supported by a grid of columns. At the base of the structure, there are four circular markers labeled A, B, C, and D. A red arrow points from marker A towards the right, and a green arrow points from marker B towards the left. The structure is shown on a light blue grid background.

The plan, section and 3D of Building B is shown **Figure 4.4**, **Figure 4.5** and **Figure 4.6**.



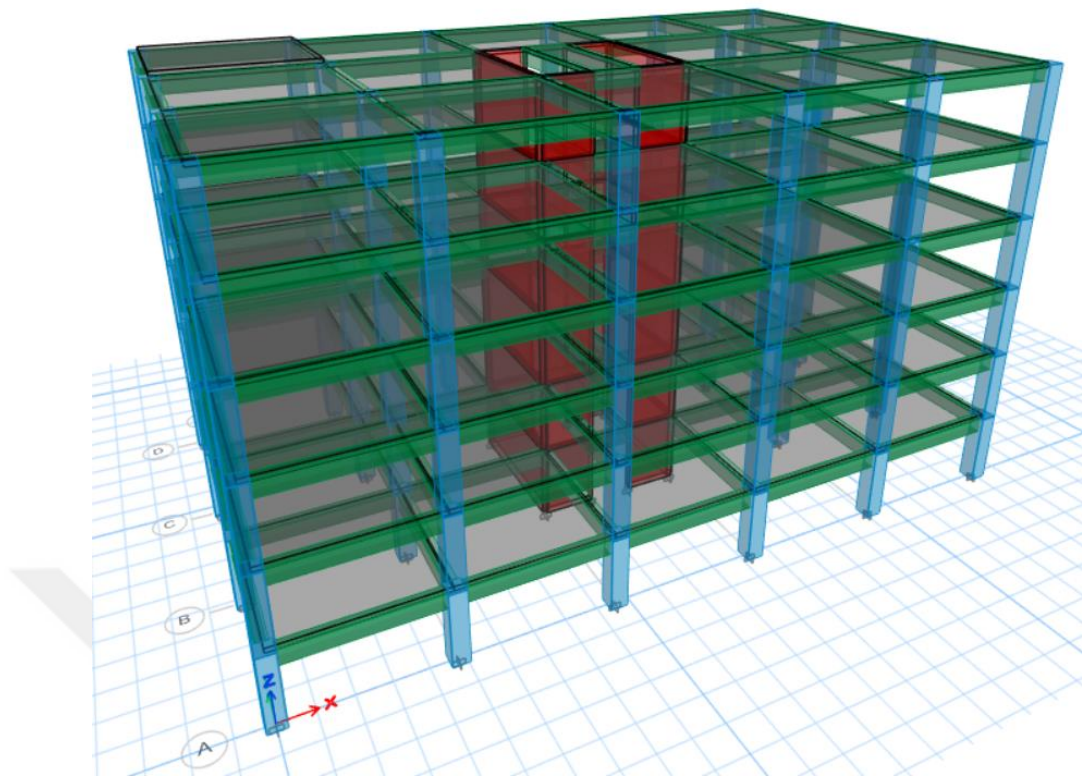


Figure 4.6: Building B ETABS model view.

The plan, section and 3D of Building C is shown **Figure 4.1**, **Figure 4.2** and **Figure 4.3**.

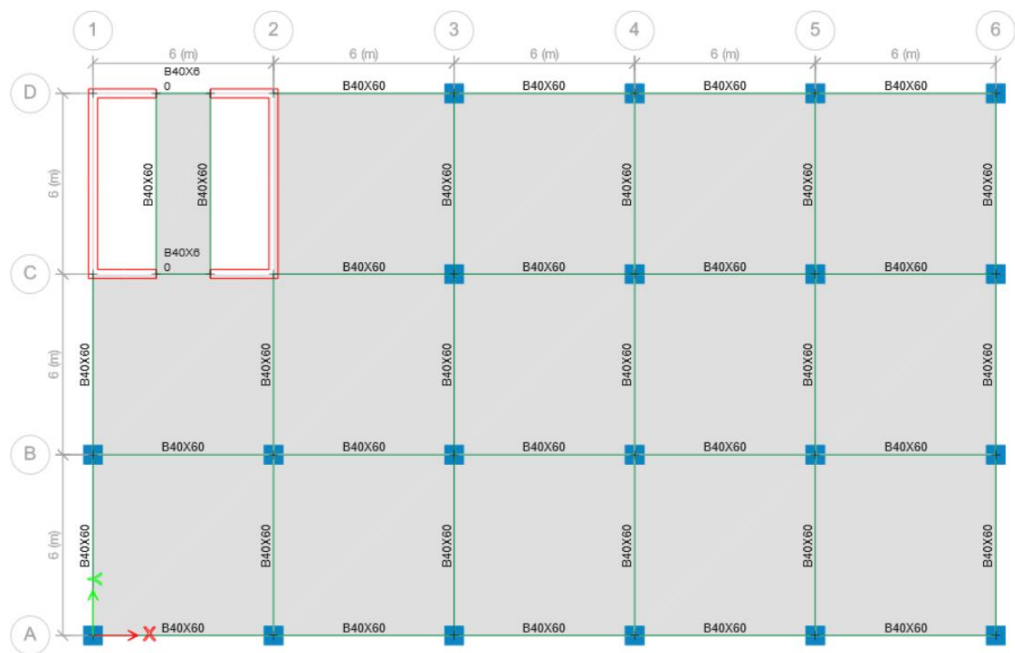


Figure 4.7: Building C ETABS typical story plan.



Figure 4.8: Building C ETABS building section.

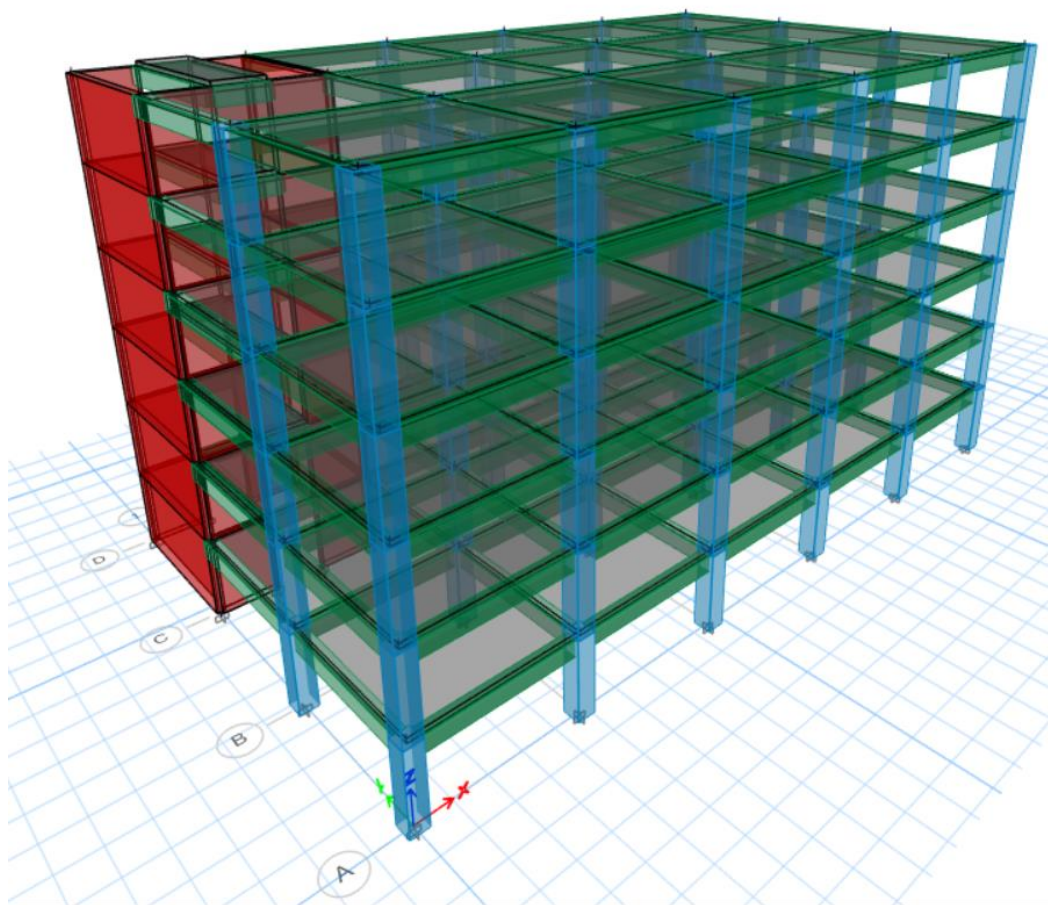


Figure 4.9: Building C ETABS model view.

The information regarding the structural elements and materials of the buildings is provided below.

- Concrete class → C30
- Rebar class → S420
- Columns dimension → 65x65 (cm)
- Columns longitudinal rebar → 16Ø20
- Columns confinement rebar → Ø12/10- 4 number
- Beams dimension → 40x60 (cm)
- Beams longitudinal rebar → 4Ø20 (top rebar)- 4Ø20 (bottom rebar)
- Beams confinement rebar → Ø12/10- 2 ties
- Slabs thickness → 17 cm
- Wall thickness→ 30 (cm)
- Wall dimentions → 30x210 (cm)- 30x600 (cm)
- Wall end zone confinement bar → Ø10/10- 2 ties
- Wall end zone longidutional bar → Ø16/12.5
- Wall web zone longidutional bar → Ø12/20
- Wall web zone transverse bar → Ø12/15

Figure 4.10 and **Figure 4.11** show column and beam section information used in the analysis models.

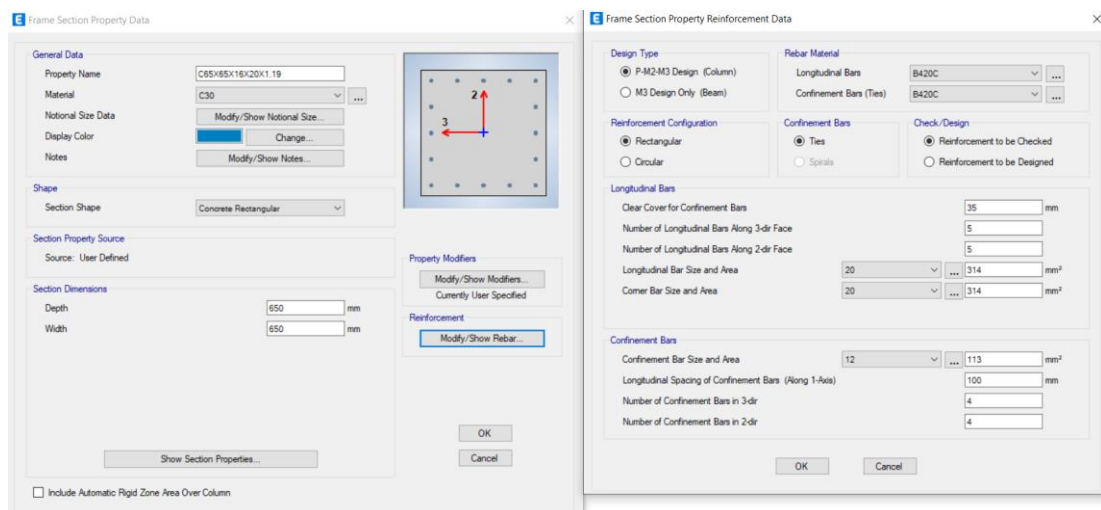


Figure 4.10: Column section informations.

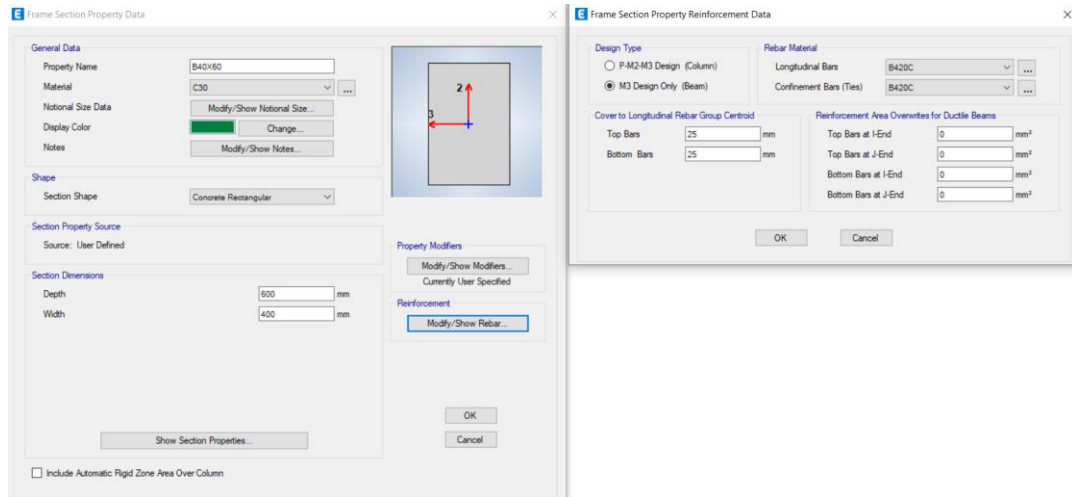


Figure 4.11: Beam section informations.

A 3D mathematical model was created for the entire building in accordance with the 2018 Turkish Building Earthquake Code (TBEC). Columns and beams were modeled as frame (bar) elements, while shear walls and slabs were modeled as shell elements. For the shear walls, only the flexural behavior in the stiff direction was considered. Semi-rigid diaphragm behavior was assumed for the floor slabs.

4.1.2. Seismic Design Parameters

For all three buildings in which nonlinear behavior is to be investigated, the analysis parameters obtained in accordance with TBEC 2018 are illustrated below using figures taken from the same code. The local soil class is assumed to be ZC (**Table 4.1**).

Table 4.1: Soil class [2].

Soil Class	Soil Type	Average in the upper 30 meters		
		(V _s) ₃₀ [m/s]	(N ₆₀) ₃₀ [pulse /30 cm]	(c _u) ₃₀ [kPa]
ZA	Strong, hard rocks	> 1500	–	–
ZB	Weak altered, medium strong rocks	760 – 1500	–	–
ZC	Very stiff sand, gravel and hard clay layers or latered, very cracked weak rocks	360 – 760	> 50	> 250
ZD	Medium-hard sand, gravel or very hard clay layers	180 – 360	15 – 50	70 – 250
ZE	Soft sand, gravel or soft-hard clay layers or profiles containing a soft clay layer (c _u < 25 kPa) of a total thickness of 3 meters providing conditions of PI > 20 and w > 40%	< 180	< 15	< 70

According to purpose of use the building building use class is 3 and building importance coefficient is 1 (**Table 4.2**). Also live load mass participating coefficient is 0.3 (**Table 4.3**). Effective section stiffness factor were used according to **Table 4.4**.

Table 4.2: Building use class and importance factor [2].

Building Use Class	Purpose of Use of the Building	Building Importance Coefficient (I)
BKS = 1	Buildings that need to be used after the earthquake, buildings where people are long-term and intensive, buildings where valuable goods are stored and buildings containing dangerous materials a) Buildings that need to be used immediately after the earthquake (Hospitals, dispensaries, health centers, fire brigade buildings and facilities, PTT and other communication facilities, transportation stations and terminals, energy production and distribution facilities, province, district governorship and municipality administration buildings, first aid and disaster planning stations) b) Schools, other educational buildings and facilities, dormitories and dormitories, military barracks, prisons, etc. c) Museums & Museums d) Buildings where toxic, explosive, flammable, etc. substances with properties are located or stored	1.5
BKS = 2	Buildings where people are located for a short time and intensively are shopping malls, sports facilities, cinemas, theaters, concert halls, places of worship, etc.	1.2
BKS = 3	Other buildings that do not fall within the definitions given for BKS=1 and BKS=2 (Residences, workplaces, hotels, building-type industrial structures, etc.)	1.0

Table 4.3: Live load mass participation coefficient [2].

Purpose of the Building	n
Warehouse etc.	0.8
School, student dormitory, sports facility, cinema, theatre, concert hall, place of worship, restaurant, store etc.	0.60
House, workplace, hotel, hospital, parking lot, etc.	0.3

Table 4.4: Effective section stiffness factors for RC [2].

Reinforced Concrete Carrier System Element	Effective Section Stiffness	
Shear Wall – Slab (in plane)	Axial	Shear
Shear Wall	0.5	0.5
Basement Shear Wall	0.8	0.5
Slab	0.25	0.25
Shear Wall – Slab (out of plane)	Flexure	Shear
Shear Wall	0.25	1
Basement Shear Wall	0.5	1
Slab	0.25	1
Bar element	Flexure	Shear
Transverse Beam	0.15	1
Frame Beam	0.35	1
Frame Column	0.7	1
Shear Wall (equivalent bar)	0.5	0.5

4.1.3. Earthquake Records

For the earthquake record, data from the February 6, 2023 Pazarcık Earthquake was obtained from AFAD's station number TK-2718. The acceleration-time graphs from both earthquake directions were used as the basis for the nonlinear time history analysis. The buildings are assumed to be located at the same coordinates as this station. Details of station and earthquake are shown **Figure 4.12**, **Table 4.5** and **Table 4.6**.

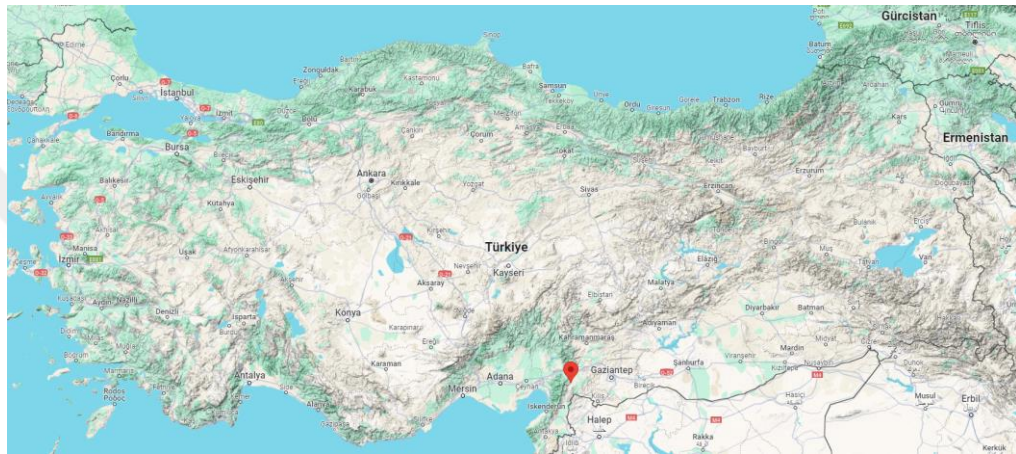


Figure 4.12: Location of TK-2718.

Table 4.5: Details about building location.

Station code	TK-2718
Province	Gaziantep
District	Islahiye
Latitude	37.00777
Longitude	36.6266
VS ₃₀ (m/sn)	572
Soil class	ZC

Table 4.6: Earthquake records details.

Earthquake Case Name	EQ-1
Earthquake Place	Pazarcık
Magnitude	M7.8
Date	6 February 2023
Hour	01:17:32
Latitude of Earthquake	37.288
Longitude of Earthquake	37.043
Depth of Earthquake (km)	8.6

Considering the local soil class parameters at the corresponding station, the acceleration-time history of the earthquake was obtained using the SeismoMatch software as shown **Figure 4.13** and **Figure 4.14**.

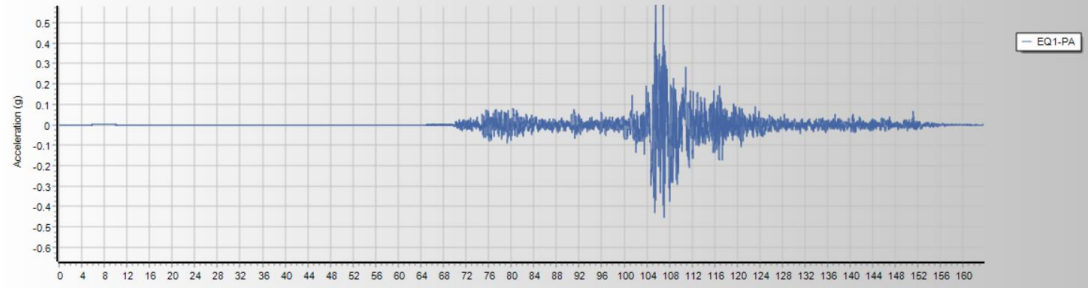


Figure 4.13: TK-2718 EW direction acceleration time.

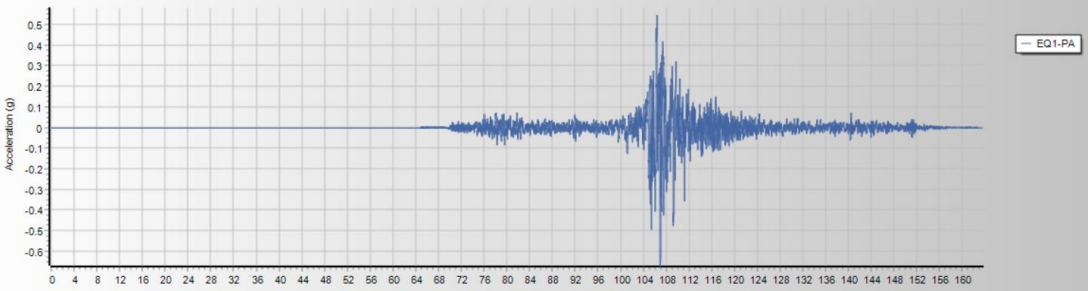


Figure 4.14: TK-2718 NS direction acceleration time.

Using the same software, when the acceleration-time history is superimposed with the elastic design acceleration spectrum, the resulting visuals for each direction are obtained separately as shown **Figure 4.15**, **Figure 4.16** and **Figure 4.17**.

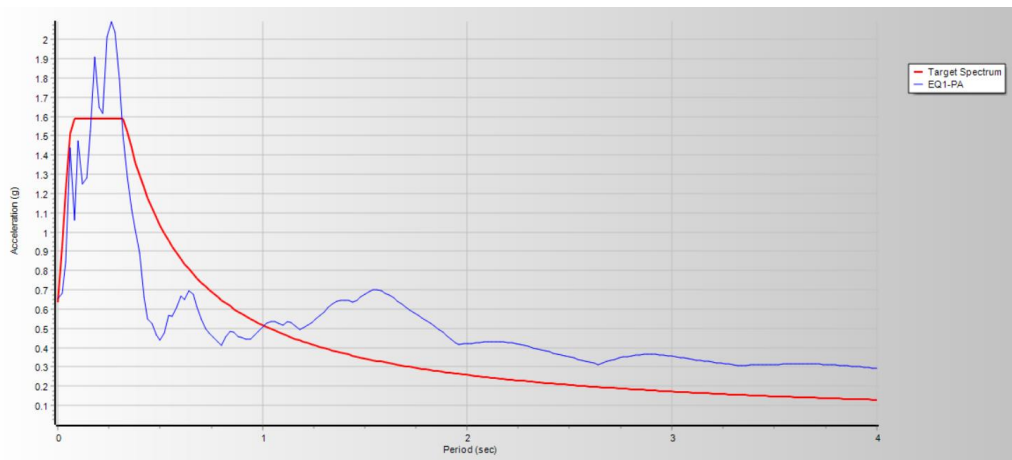


Figure 4.15: TK-2718 NS elastic design spectrum superposition.

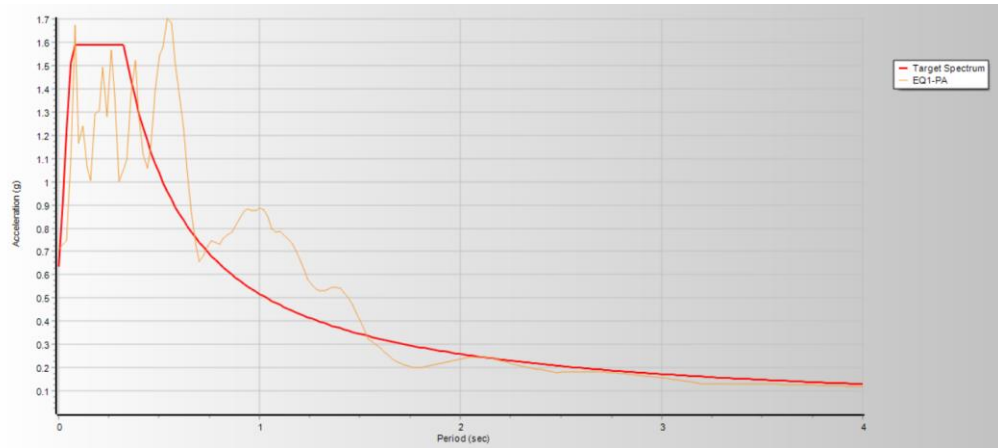


Figure 4.16: TK-2718 EW elastic design spectrum superposition.

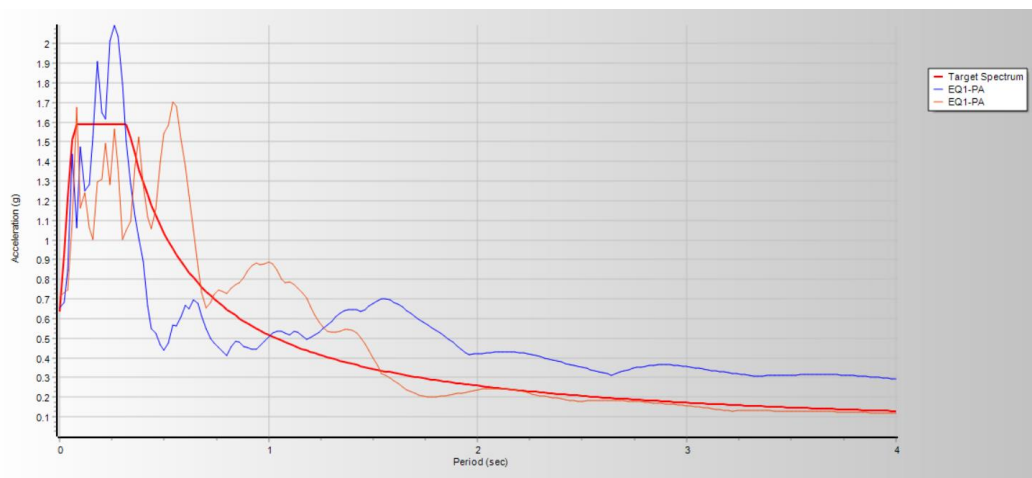


Figure 4.17: TK-2718 all directions elastic design spectrum superposition.

4.1.4. Beam Plastic Hinge Properties

As stated in Chapter 3, the nonlinear behavior of beams was modeled using the lumped plasticity approach. For this purpose, plastic hinges were defined at the ends of beams where plastic rotation is likely to occur. These plastic hinges represent the regions where earthquake energy will be dissipated within the beams. The software XTRACT was used for this modeling. Since the reinforcement areas on the top and bottom faces of the beams are not the same, the plastic behavior of the same cross-section will differ under cyclic loading. Therefore, two separate moment-curvature analyses must be performed for each beam section—one for the positive direction (**Figure 4.18**) and one for the negative direction (**Figure 4.19**).

XTRACT Analysis Report -

For use only in an academic or research setting.

Section Name: B40X60X4Q20-4Q16

15.04.2025

Loading Name: POSITIVE

B40X60X4Q20-4Q16

Analysis Type: Moment Curvature

Page __ of __

Section Details:

X Centroid: -8.012E-9 m
Y Centroid: 2.978E-3 m
Section Area: .2400 m²

Loading Details:

Incrementing Loads: Mxx Only
Number of Points: 31
Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Steel
Failure Strain: 80.00E-3 Tension
Curvature at Initial Load: 0 1/m
Curvature at First Yield: 5.002E-3 1/m
Ultimate Curvature: .1617 1/m
Moment at First Yield: 269.1 kN-m
Ultimate Moment: 355.4 kN-m
Centroid Strain at Yield: .8393E-3 Ten
Centroid Strain at Ultimate: 39.24E-3 Ten
N.A. at First Yield: .1678 m
N.A. at Ultimate: .2426 m
Energy per Length: 52.05 kN
Effective Yield Curvature: 5.584E-3 1/m
Effective Yield Moment: 300.4 kN-m
Over Strength Factor: 1.183
Plastic Rotation Capacity: 46.85E-3 rad
EI Effective: 5.38E+7 N-m²
Yield EI Effective: 352.4E+3 N-m²
Bilinear Hardening Slope: .6552 %
Curvature Ductility: 28.97

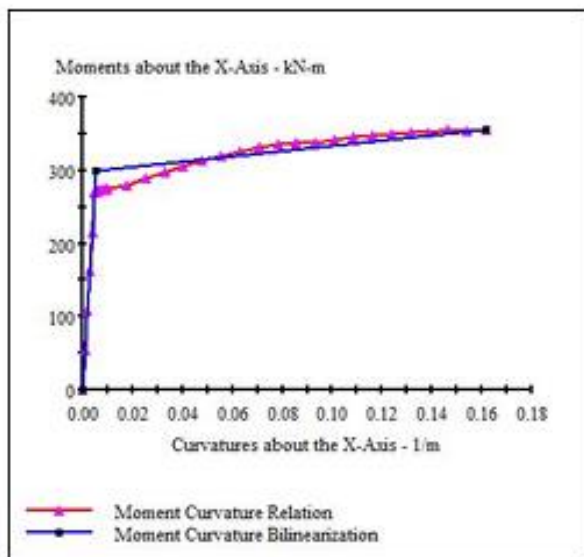
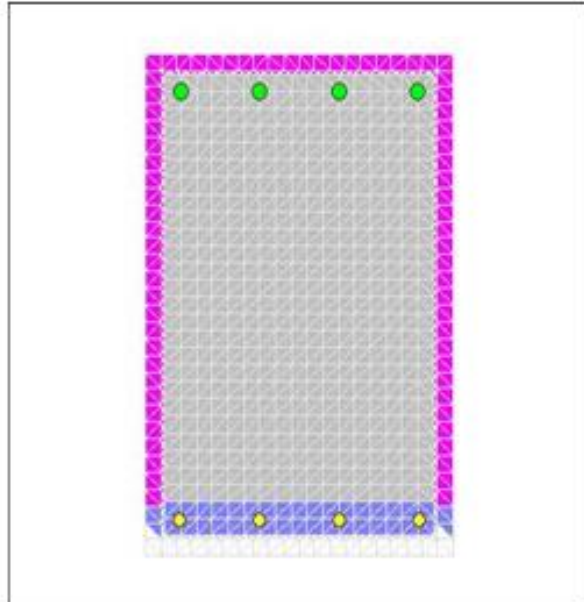


Figure 4.18: XTract moment curvature analysis for positive direction.

XTRACT Analysis Report -

For use only in an academic or research setting.

Section Name: B40X60X4Q20-4Q16

Loading Name: NEGATIVE

Analysis Type: Moment Curvature

15.04.2025

B40X60X4Q20-4Q16

Page __ of __

Section Details:

X Centroid: -8.012E-9 m

Y Centroid: 2.978E-3 m

Section Area: .2400 m²

Loading Details:

Incrementing Loads: Mxx Only

Number of Points: 31

Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: Steel

Failure Strain: 80.00E-3 Tension

Curvature at Initial Load: 0 1/m

Curvature at First Yield: -4.654E-3 1/m

Ultimate Curvature: -.1577 1/m

Moment at First Yield: -174.1 kN-m

Ultimate Moment: -230.6 kN-m

Centroid Strain at Yield: .8901E-3 Ten

Centroid Strain at Ultimate: 39.00E-3 Ten

N.A. at First Yield: -.1913 m

N.A. at Ultimate: -.2472 m

Energy per Length: 33.16 kN

Effective Yield Curvature: 5.284E-3 1/m

Effective Yield Moment: 197.6 kN-m

Over Strength Factor: -1.167

Plastic Rotation Capacity: -48.90E-3 rad

EI Effective: 3.74E+7 N-m²

Yield EI Effective: 216.5E+3 N-m²

Bilinear Hardening Slope: .5787 %

Curvature Ductility: 29.85

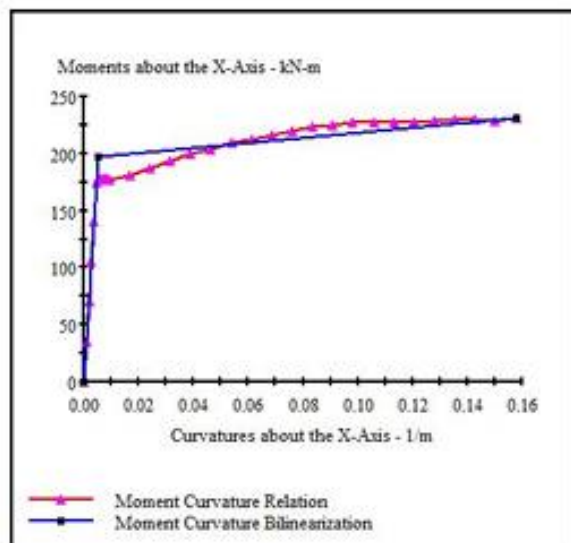
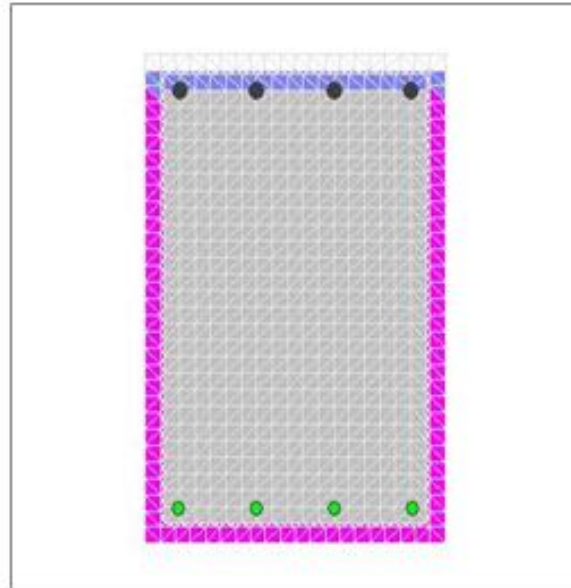


Figure 4.19: XTract moment curvature analysis for negative direction.

Based on the data obtained from the moment-curvature analysis, the properties of the regions at the beam ends that are expected to exhibit plastic behavior were input into the ETABS software as shown **Figure 4.20**.

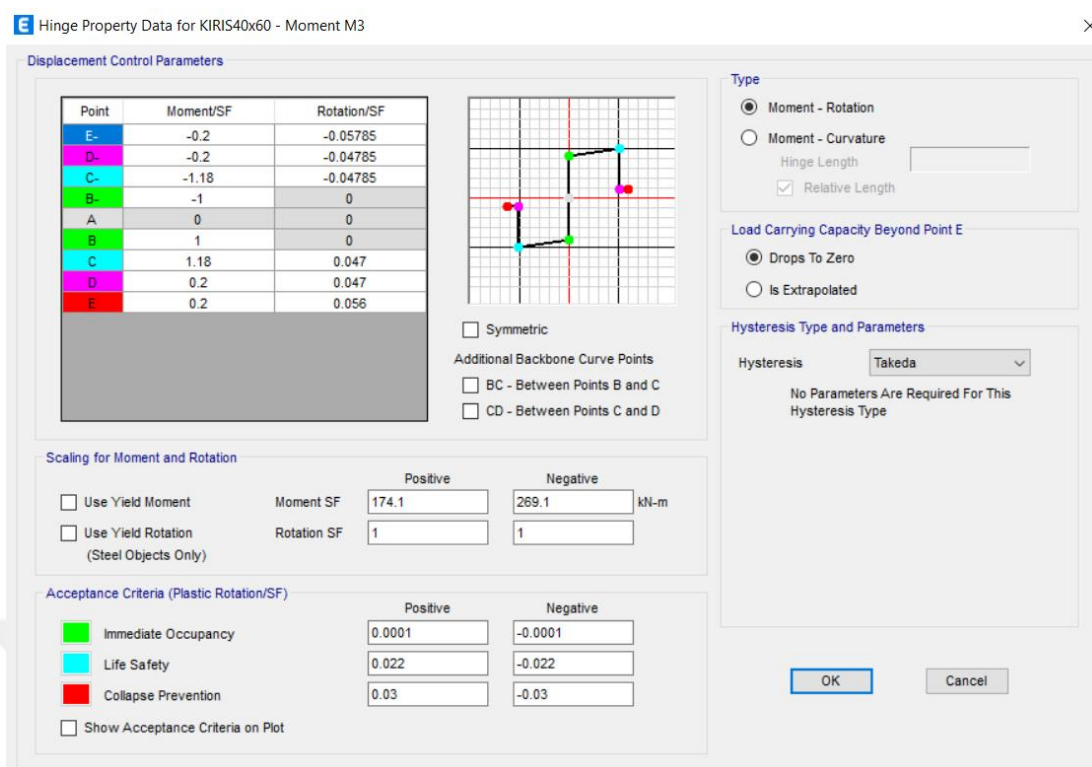


Figure 4.20: M3 hinge definition for beam.

4.1.5. Column Fiber Hinges Definitions

As stated in Chapter 3, the nonlinear behavior of columns was modeled using the distributed plasticity approach. For this purpose, the column cross-section was divided into fiber elements, including confined and unconfined concrete regions, as well as reinforcement bars, which are modeled as fibers capable of only axial elongation or shortening. Each reinforcement bar is represented by a fiber element, while the concrete is modeled using fiber elements with defined areas. The nonlinear behavior of the cross-section is governed by the stress-strain relationships of the materials assigned to these fibers. These fiber properties are automatically generated at the column ends within the ETABS software (**Figure 4.22**). Since the columns are subjected to biaxial bending, both bending moments in two directions and the axial force are considered in the distributed plasticity model. Therefore, the definition is made in the form of a P-M3-M2 interaction.

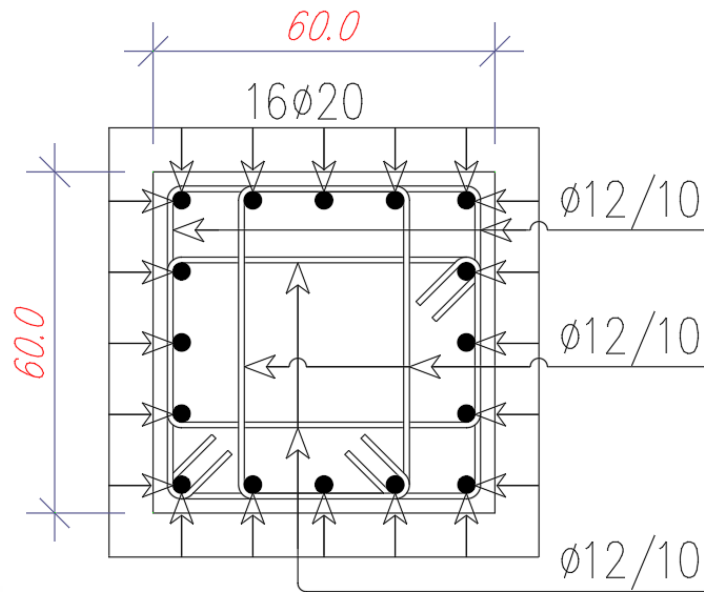


Figure 4.21: Column cross section rebar details.

E Define Fibers for Hinge C2H2 (Fiber P-M2-M3) ×

Control

☒ Overlay Frame Section on Plot

Section: C65X65X16X20X1.19

☐ Make All Fibers Gray

Fiber Definition Data

Fiber	Area cm²	Coord3 mm	Coord2 mm	Material /// Stress Strain Curve
1	3.1	268	-268	B420C /// SSC1
2	3.1	268	-134	B420C /// SSC1
3	3.1	268	0	B420C /// SSC1
4	3.1	268	134	B420C /// SSC1
5	3.1	268	268	B420C /// SSC1
6	3.1	-268	-268	B420C /// SSC1
7	3.1	-268	-134	B420C /// SSC1
8	3.1	-268	0	B420C /// SSC1
9	3.1	-268	134	B420C /// SSC1
10	3.1	-268	268	B420C /// SSC1

Sort by Coord3
Sort by Coord2
Add Fiber
Delete Selected Fibers

Show Properties...

OK
Cancel

Figure 4.22: Fiber hinge definition for column.

4.1.6. Shear Wall Fiber Hinges Definitions

Shear walls were defined using the fiber section model. For each pier of the shear walls, a P-M3 fiber section model was defined (**Figure 4.24** and **Figure 4.25**). No definition was made for the minor directions of the walls. Detail example is shown **Figure 4.26**.

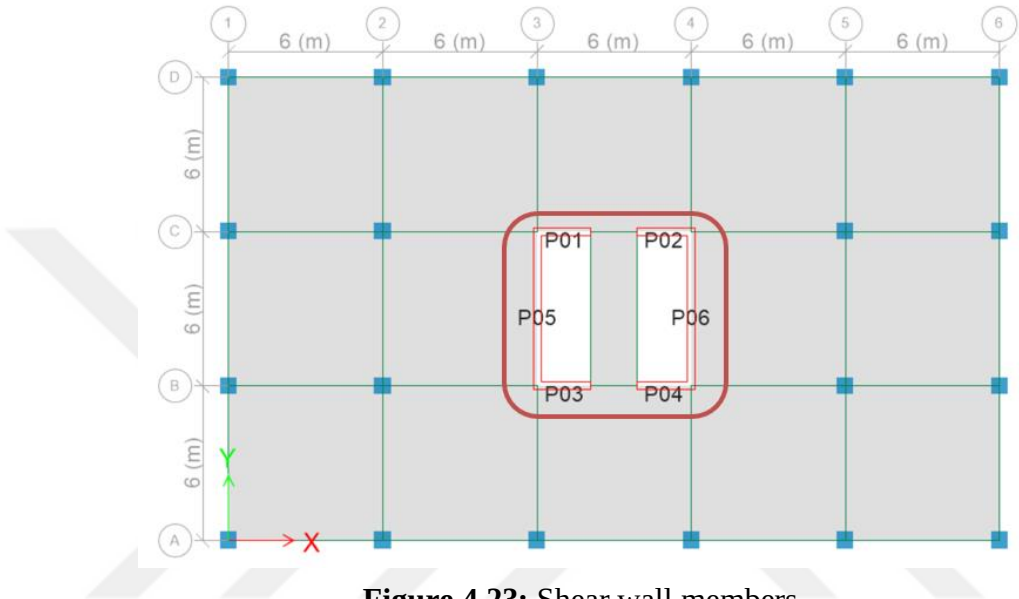


Figure 4.23: Shear wall members.

Wall Hinge Reinforcement

Wall Design Type
☒ Pier
☐ Spandrel

Rebar Material and Clear Cover
Material Flexure B420C
Material Shear B420C
Bar Clear Cover 35 mm

Rebar Layout

Geometry

Start X (mm)	Start Y (mm)	End X (mm)	End Y (mm)	Length (mm)	Thickness (mm)	Start Zone Size (mm)	End Zone Size (mm)
0	18000	2100	18000	2100	300	700	700

Reinforcement

Flexural Detail - Each Face

Station	Bar Size	Bar Spacing (mm)	Number of Bars
Start	16		4
Center	12	200	3
End	16		4

Shear/Confinement Detail

Station	Bar Size	Bar Spacing (mm)	Confined
Start	10	100	Yes
Center	12	150	No
End	10	100	Yes

Flexural Detail (Additional Individual Bars)

Material	Distance (mm)	Area (mm ²)

Figure 4.24: P1-P2-P3-P4 Fiber hinge definition for shear wall.

E Wall Hinge Reinforcement

Wall Design Type

☒ Pier

☐ Spandrel

Rebar Material and Clear Cover

Material Flexure B420C

Material Shear B420C

Bar Clear Cover 35 mm

Rebar Layout

Geometry

Start X (mm)	Start Y (mm)	End X (mm)	End Y (mm)	Length (mm)	Thickness (mm)	Start Zone Size (mm)	End Zone Size (mm)
0	12000	0	18000	6000	300	1200	1200

Reinforcement

Flexural Detail - Each Face

Station	Bar Size	Bar Spacing (mm)	Number of Bars
Start	16		10
Center	12	200	18
End	16		10

Flexural Detail (Additional Individual Bars)

Material	Distance (mm)	Area (mm ²)
*		

Shear/Confinement Detail

Station	Bar Size	Bar Spacing (mm)	Confined
Start	10	100	Yes
Center	12	150	No
End	10	100	Yes

Figure 4.25: P1-P2-P3-P4 Fiber hinge definition for shear wall.

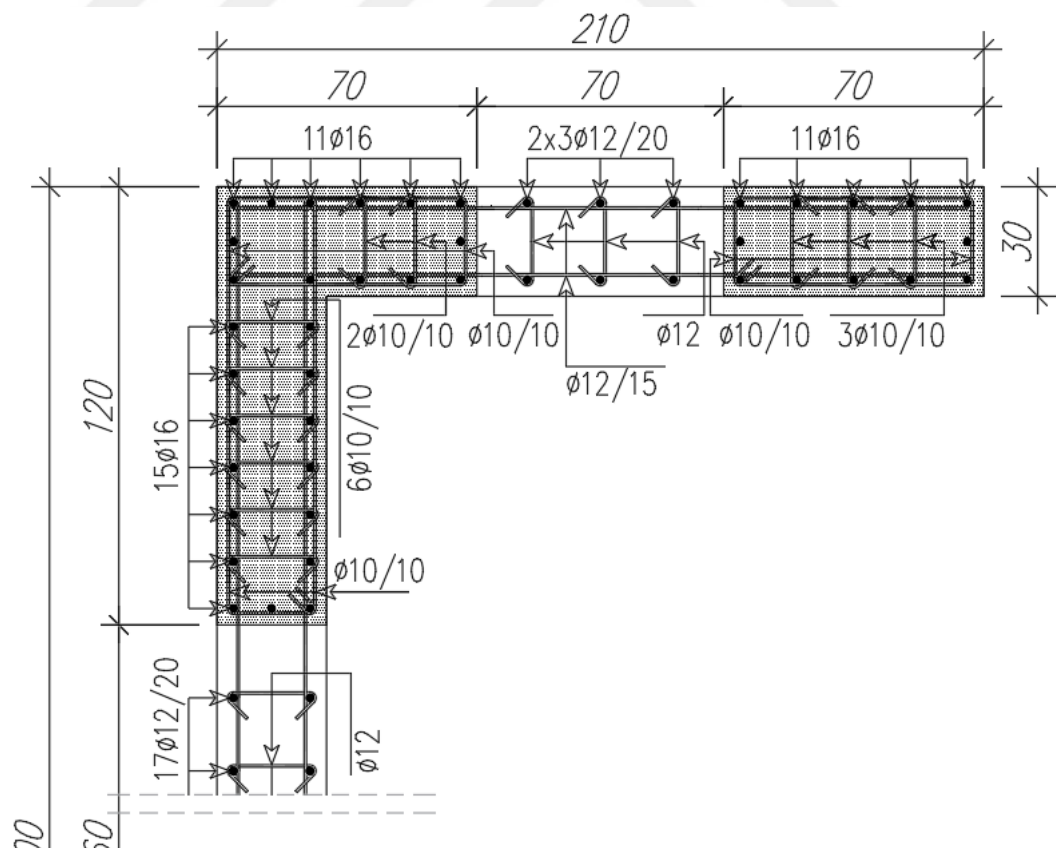


Figure 4.26: Shear wall cross section rebar details.

4.1.7. Mass Properties

The building masses used as the basis for the nonlinear analysis are presented **Table 4.7**.

Table 4.7: Mass information of buildings.

Story	Mass (KN.sec ² /m)		
	Building A	Building B	Building C
Story6	805.9	799.8	790.7
Story5	844.7	855.5	846.4
Story4	844.7	855.5	846.4
Story3	844.7	855.5	846.4
Story2	844.7	855.5	846.4
Story1	850.2	855.5	846.4
Base	38.8	55.7	55.7
Total	5073.5	5133.0	5078.3

4.2. Output of Numerical Analysis

4.2.1. Modal Analysis Results

For the buildings, a modal analysis was first conducted to determine their modal behavior characteristics. The modal analysis results used as the basis for the seismic analysis are presented **Table 4.8**.

Table 4.8: Modal participating mass ratios of buildings.

Modal Participating Mass Ratios								
Building A								
Case	Mode	Period (sec)	UX	UY	UZ	RX	RY	RZ
Modal_Tum	1	1.193	0	0.7866	0	0.2227	0	0
Modal_Tum	2	1.152	0.7898	0	0	0	0.2191	0
Modal_Tum	3	1.033	0	0	0	0	0	0.7895
Building B								
Case	Mode	Period (sec)	UX	UY	UZ	RX	RY	RZ
Modal_Tum	1	0.624	0.7614	0	0	0	0.2485	0
Modal_Tum	2	0.443	0	0.7507	0	0.2595	0	0
Modal_Tum	3	0.241	0	0	0	0	0	0.1182
Building C								
Case	Mode	Period (sec)	UX	UY	UZ	RX	RY	RZ
Modal_Tum	1	0.964	0.122	0.3935	0	0.1148	0.0355	0.2714
Modal_Tum	2	0.609	0.6351	0.1005	0	0.0329	0.2097	0.024
Modal_Tum	3	0.305	0.0053	0.001	0	0.3459	0.0589	0.2496

4.2.2. Pushover Curves

To initially understand the global behavior of these structures, pushover curves were obtained for all three buildings using the conventional single-mode pushover analysis method. In these analyses, all buildings were pushed to a target displacement of 250 mm, and the corresponding pushover curves were generated. The base shear forces that developed in the buildings at the same target displacement are shown in the graphical visuals **Figure 4.27** and **Figure 4.28**. According to these visuals, the most flexible system is Building A, which was designed as a frame system. The most rigid system is Building B, which has a shear wall-frame system with walls placed symmetrically at the center of the structure. Building C, which also has a shear wall-frame system but includes torsional irregularity, exhibits a base shear force that falls between the other two systems.

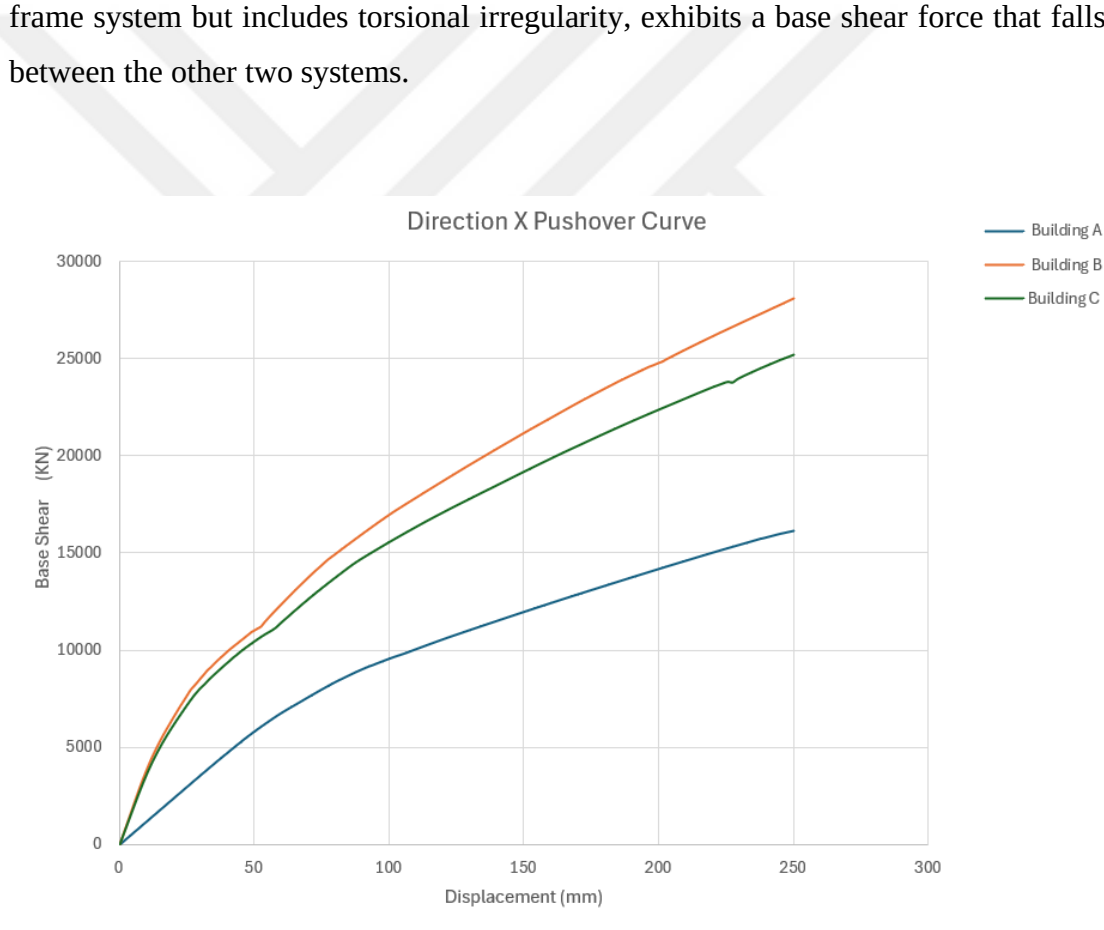


Figure 4.27: X direction pushover curves for all buildings.

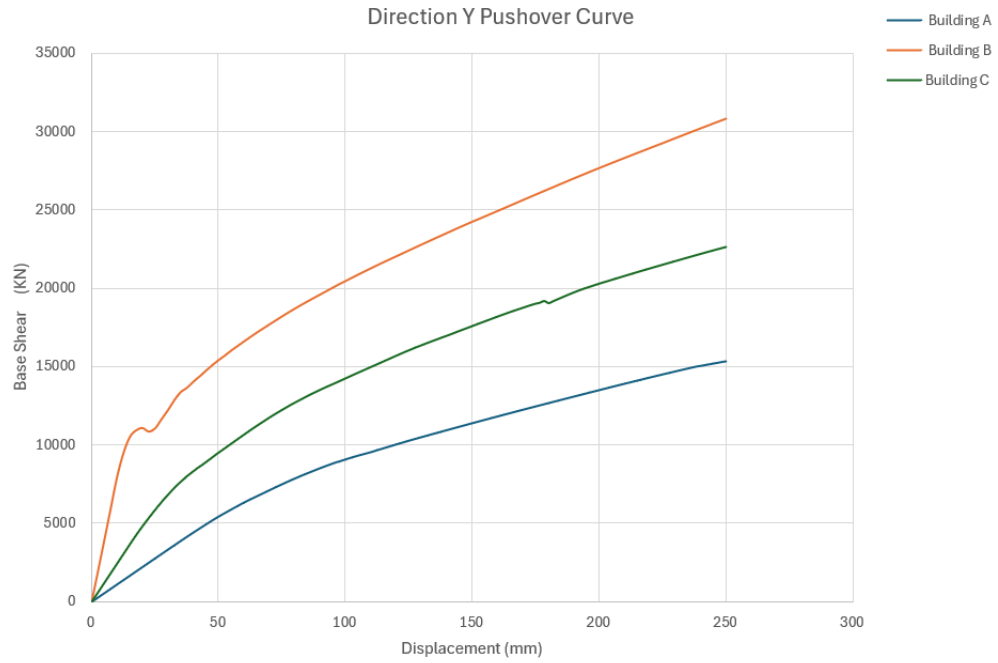


Figure 4.28: Y direction pushover curves for all buildings.

4.2.3. Story Force, Story Drift and Peak Displacement Results

According to the Nonlinear Time History Analysis results, the story forces (**Figure 4.29** and **Figure 4.30**), interstory drift values (**Figure 4.31** and **Figure 4.32**) and roof center of mass displacements (**Figure 4.33** and **Figure 4.34**) are presented in the below.

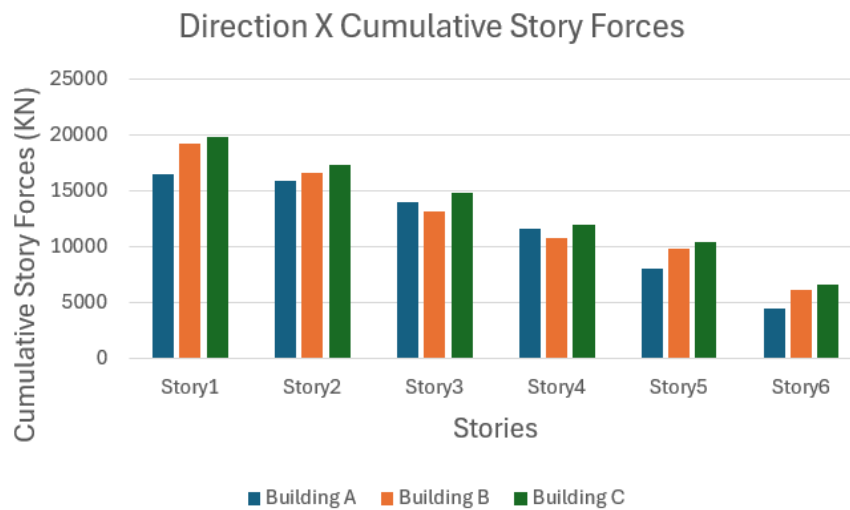


Figure 4.29: Cumulative story forces results (Direction X).

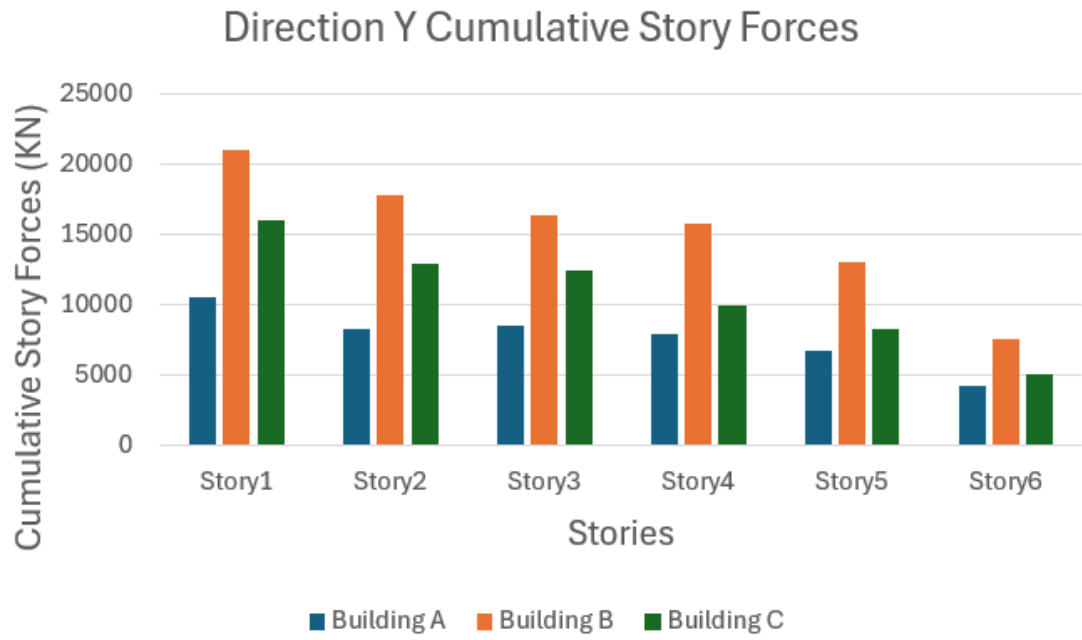


Figure 4.30: Cumulative story forces results (Direction Y).

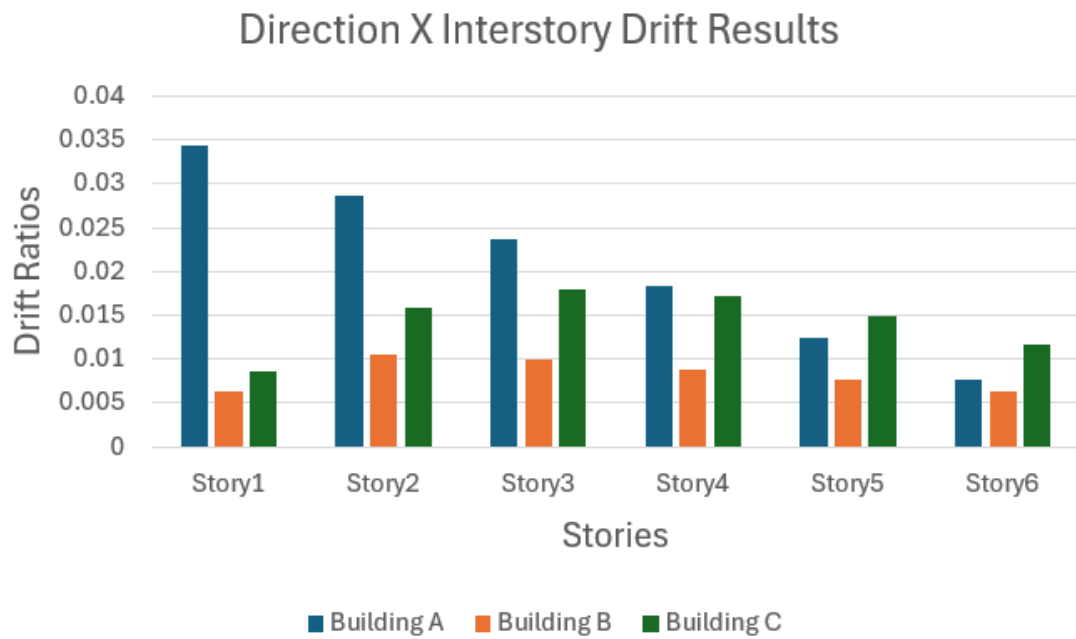


Figure 4.31: Interstory drift ratio results (Direction X).

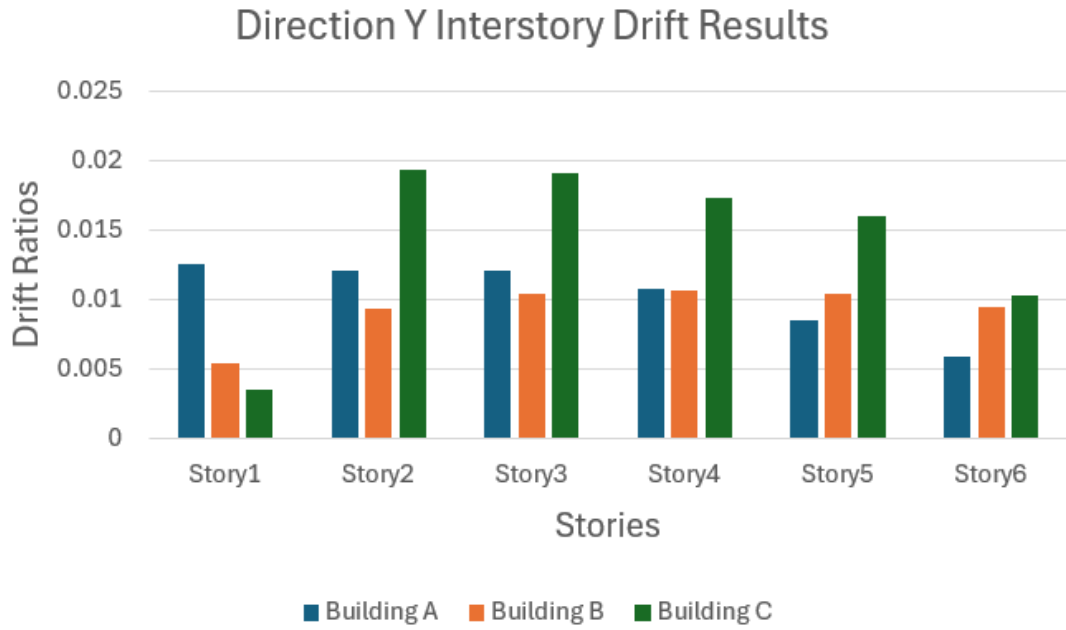


Figure 4.32: Interstory drift ratio results (Direction Y).

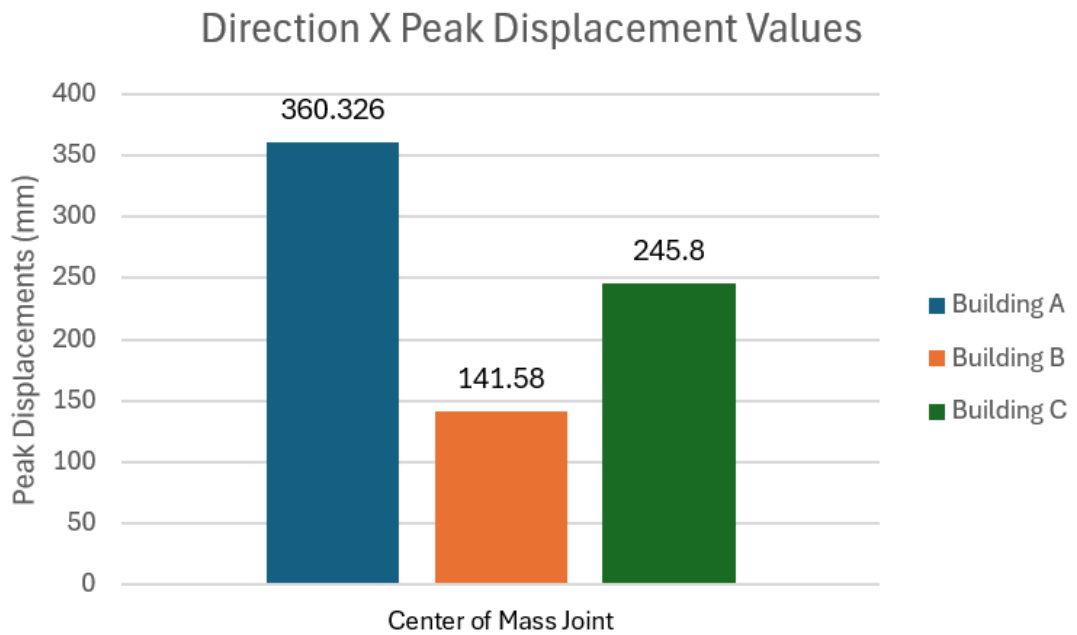


Figure 4.33: Story 6 center of mass displacement results (Direction X).

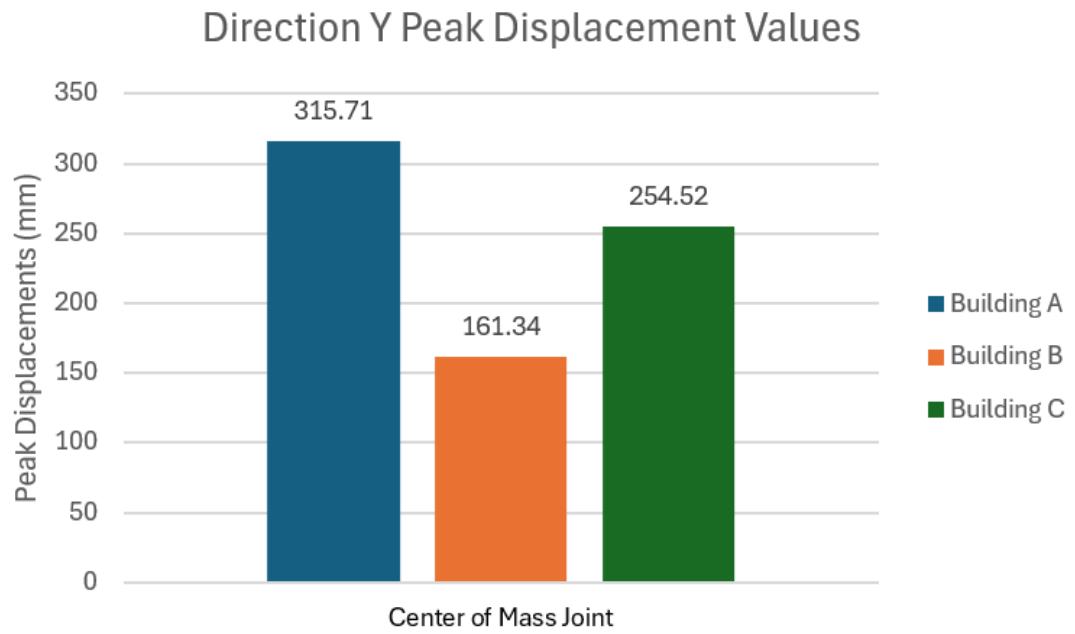


Figure 4.34: Story 6 center of mass displacement results (Direction Y).

4.2.4. Building A Beam Plastic Rotation Results

According to the 2018 Turkish Building Earthquake Code (TBEC), the plastic rotation formula for beams under the collapse prevention performance level is given below.

Also **Figure 4.35** shows label of beams for Building A.

$$\theta_p (CP) = \frac{2}{3} * \left[(\Phi_u - \Phi_y) * L_p * \left(1 - 0.5 * \frac{L_p}{L_s} \right) + 4.5 * \Phi_u * db \right] \quad (4.1)$$

$$\theta_p (LS) = 0.75 * \theta_p (CP) \quad (4.2)$$

$$\theta_p (IO) = 0 \quad (4.3)$$

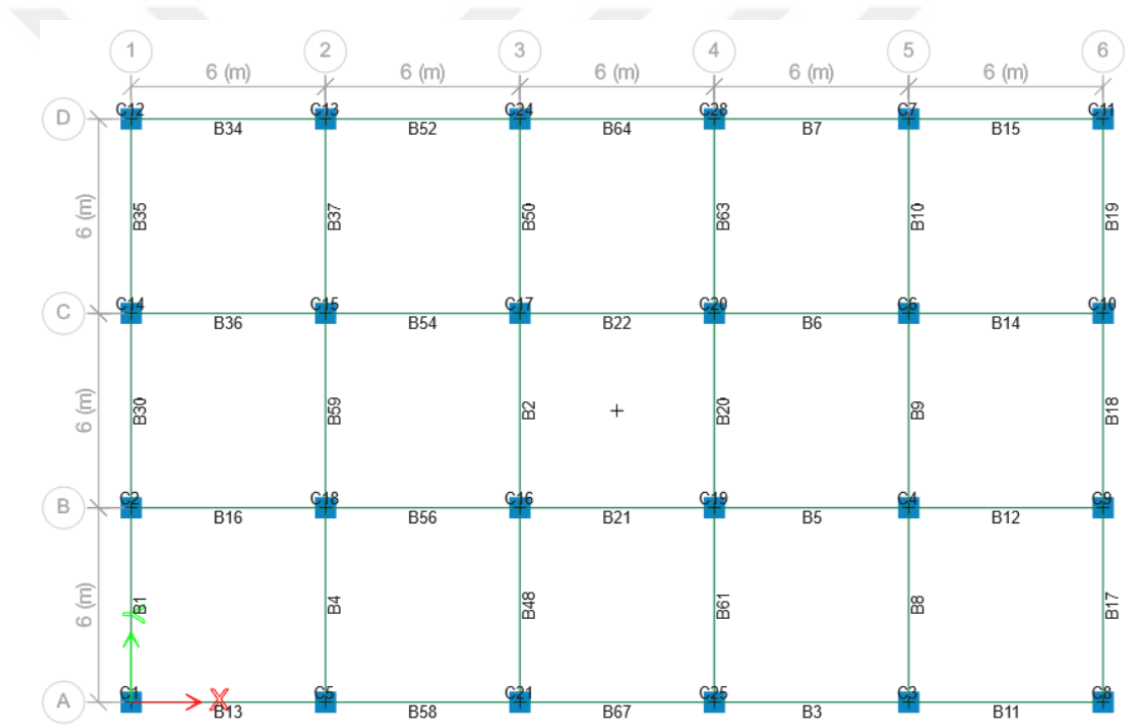


Figure 4.35: Building A Beam Labels.

The table in Appendix-A lists the damage states corresponding to ductile behavior for beams on a single floor.

4.2.5. Building B Beam Plastic Rotation Results

According to the 2018 Turkish Building Earthquake Code (TBEC), the plastic rotation formula for beams under the collapse prevention performance level is given below.

Also **Figure 4.36** shows label of beams for Building B.

$$\theta_p (CP) = \frac{2}{3} * \left[(\Phi_u - \Phi_y) * L_p * \left(1 - 0.5 * \frac{L_p}{L_s} \right) + 4.5 * \Phi_u * db \right] \quad (4.1)$$

$$\theta_p (LS) = 0.75 * \theta_p (CP) \quad (4.2)$$

$$\theta_p (IO) = 0 \quad (4.3)$$

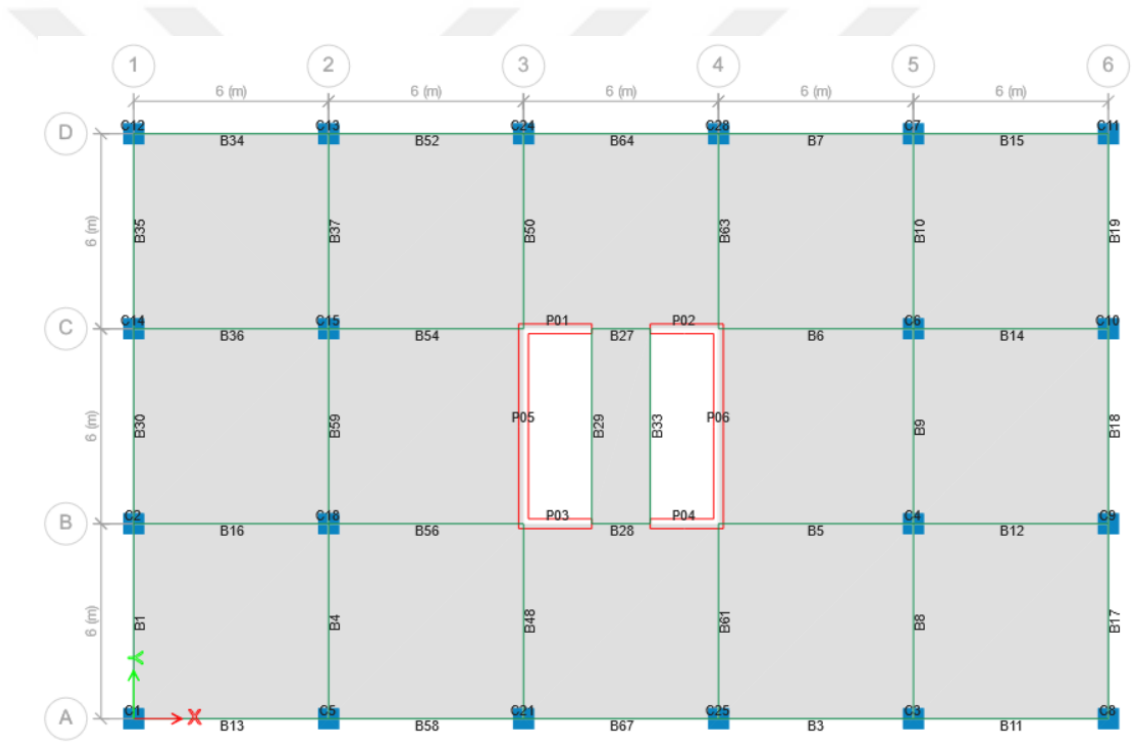


Figure 4.36: Building B Beam Labels.

The table in Appendix-B lists the damage states corresponding to ductile behavior for beams on a single floor.

4.2.6. Building C Beam Plastic Rotation Results

According to the 2018 Turkish Building Earthquake Code (TBEC), the plastic rotation formula for beams under the collapse prevention performance level is given below.

Also **Figure 4.37** shows label of beams for Building C.

$$\theta_p (CP) = \frac{2}{3} * \left[(\Phi_u - \Phi_y) * L_p * \left(1 - 0.5 * \frac{L_p}{L_s} \right) + 4.5 * \Phi_u * db \right] \quad (4.1)$$

$$\theta_p (LS) = 0.75 * \theta_p (CP) \quad (4.2)$$

$$\theta_p (IO) = 0 \quad (4.3)$$

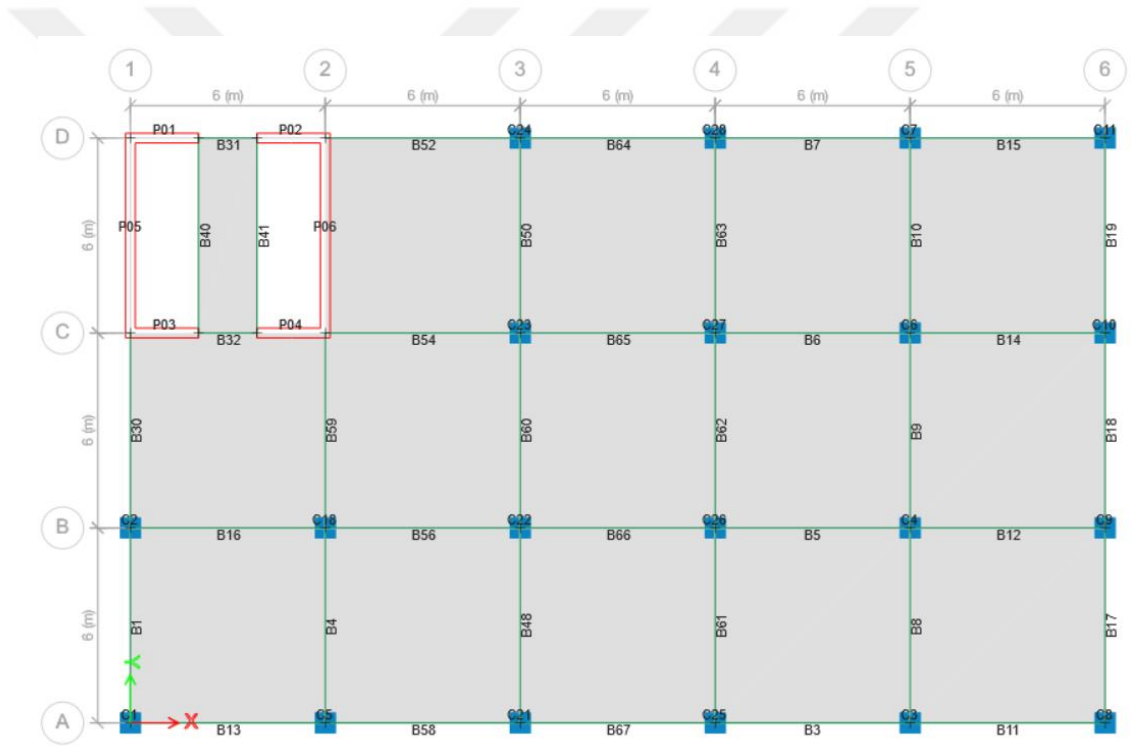


Figure 4.37: Building C Beam Labels.

The table in Appendix-C lists the damage states corresponding to ductile behavior for beams on a single floor.

4.2.7. Summary Table for Beam Ductile Behaviour Results

Based on all analyses, the **Figure 4.38** presents the damage levels observed in the beams for each of the three building types.

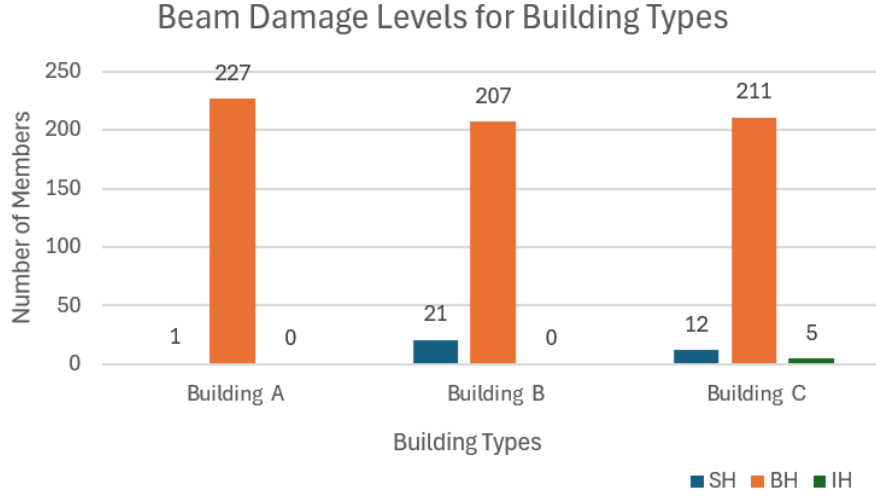


Figure 4.38: Summary graph figure for damage levels for beams.

4.2.8. Column Fiber Strain Results

According to the 2018 Turkish Building Earthquake Code (TBEC), the strain limits and corresponding damage states for concrete and reinforcement fibers are given below. For columns, beams, and shear walls with rectangular cross-sections, the collapse strain limit is defined by the formula shown below.

$$\varepsilon_c(CP) = 0.0035 + 0.04\sqrt{wwe} \quad (4.4)$$

$$\varepsilon_c(LS) = 0.75 * \varepsilon_c(CP) \quad (4.5)$$

$$\varepsilon_c(IO) = 0.0025 \quad (4.6)$$

The strain limits for reinforcement steel are also defined by the formula given below.

$$\varepsilon_s(CP) = 0.4\varepsilon_{su} \quad (4.7)$$

$$\varepsilon_s(LS) = 0.75\varepsilon_s(CP) \quad (4.8)$$

$$\varepsilon_s(I0) = 0.0075 \quad (4.9)$$

According to these limit values, the damage state table for Building A, Building B and Building C is presented in Appendix-A, Appendix-B and Appendix-C.

4.2.9. Summary Graph for Column Ductile Behaviour

Based on all analyses, the **Figure 4.39** presents the damage levels observed in the columns for each of the three building types.

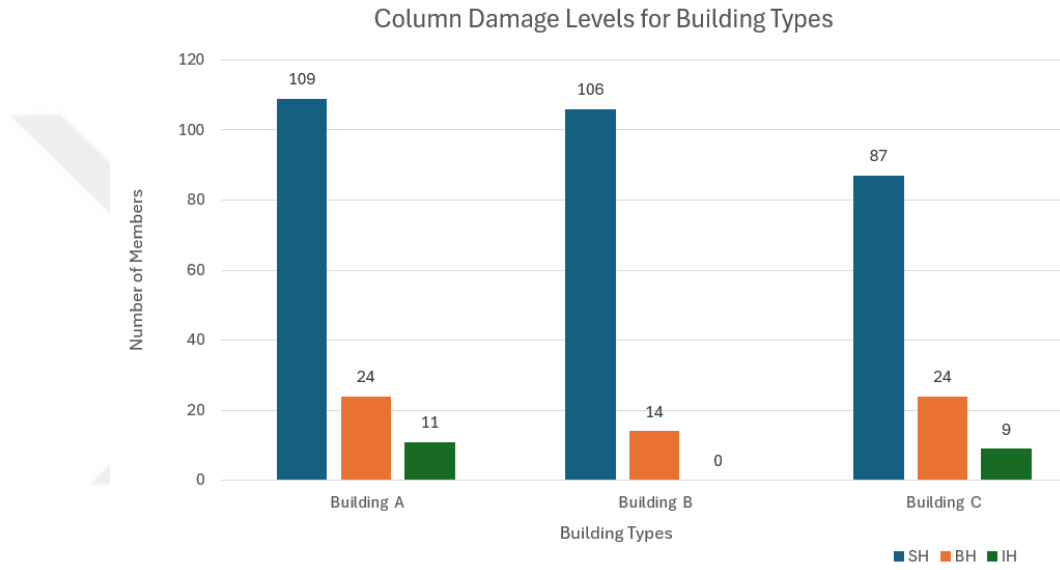


Figure 4.39: Summary graph figure for damage levels for columns.

4.2.10. Building B Shear Wall Fiber Strain Results

Based on all analyses, the strain values and corresponding damage states in the concrete and reinforcement fibers of the shear walls in Building B are presented in the **Table 4.9**.

Table 4.9: Fiber strain results for Building B.

Story	Label	Concrete Strain	Rebar Strain	$\epsilon_c(\text{CP})$	$\epsilon_c(\text{LS})$	$\epsilon_c(\text{IO})$	$\epsilon_s(\text{CP})$	$\epsilon_s(\text{LS})$	$\epsilon_s(\text{IO})$	Concrete Result	Rebar Result
Story6	W1	0.00029	0.00027	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W2	6.6E-05	6.4E-05	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W4	0.0003	0.00028	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W5	0.00021	0.00035	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W6	5.5E-05	5.4E-05	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W10	0.00022	0.00021	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W1	0.0006	0.00057	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W2	0.00014	0.00014	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W4	0.00066	0.00093	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W5	0.00097	0.00092	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W6	0.00012	0.00012	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W10	0.00094	0.0009	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W1	0.00111	0.00106	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W2	0.00018	0.00018	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W4	0.0008	0.0011	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W5	0.00115	0.00105	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W6	0.00022	0.00021	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W10	0.00086	0.00082	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W1	0.00081	0.00078	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W2	0.0003	0.00029	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W4	0.00151	0.00144	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W5	0.00064	0.00124	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W6	0.00037	0.00036	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W10	0.00076	0.00072	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W1	0.00056	0.00054	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W2	0.00048	0.00046	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W4	0.00065	0.00082	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W5	0.00084	0.00081	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W6	0.0006	0.00058	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W10	0.00082	0.00079	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story1	W1	0.0097	0.01762	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH
Story1	W2	0.00933	0.01689	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH
Story1	W4	0.00803	0.0129	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH
Story1	W5	0.00953	0.01223	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH
Story1	W6	0.009	0.01458	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH
Story1	W10	0.00903	0.01706	0.014	0.011	0.003	0.03	0.02	0.008	BH	BH

4.2.11. Building C Shear Wall Fiber Strain Results

Based on all analyses, the strain values and corresponding damage states in the concrete and reinforcement fibers of the shear walls in Building C are presented in the **Table 4.10**.

Table 4.10: Fiber strain results for Building C.

Story	Label	Concrete Strain	Rebar Strain	$\epsilon_c(\text{CP})$	$\epsilon_c(\text{LS})$	$\epsilon_c(\text{IO})$	$\epsilon_s(\text{CP})$	$\epsilon_s(\text{LS})$	$\epsilon_s(\text{IO})$	Concrete Result	Rebar Result
Story6	W7	3.7E-05	5.7E-05	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W8	7.3E-05	7.2E-05	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W9	0.00028	0.00027	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W11	0.00036	0.00034	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W12	0.00022	0.00021	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story6	W14	0.00033	0.00032	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W7	0.00012	0.00013	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W8	0.00015	0.00027	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W9	0.0007	0.00149	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W11	0.00081	0.00082	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W12	0.00051	0.00066	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story5	W14	0.00068	0.00074	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W7	0.00019	0.00096	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W8	0.0002	0.00065	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W9	0.00115	0.00172	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W11	0.0011	0.00207	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W12	0.00101	0.00186	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story4	W14	0.00086	0.00222	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W7	0.00022	0.00034	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W8	0.00029	0.00028	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W9	0.00101	0.00125	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W11	0.00076	0.00072	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W12	0.00089	0.00085	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story3	W14	0.00057	0.00054	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W7	0.00054	0.00221	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W8	0.00052	0.00179	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W9	0.00054	0.00322	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W11	0.00055	0.0028	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W12	0.00056	0.00271	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story2	W14	0.00053	0.00397	0.014	0.011	0.003	0.03	0.02	0.008	SH	SH
Story1	W7	0.00171	0.01116	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH
Story1	W8	0.00177	0.01418	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH
Story1	W9	0.00181	0.0154	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH
Story1	W11	0.0017	0.01124	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH
Story1	W12	0.00176	0.01746	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH
Story1	W14	0.00227	0.01381	0.014	0.011	0.003	0.03	0.02	0.008	SH	BH

4.2.12. Building A Beam Shear Force Check

For brittle failure evaluation, shear force demands were compared with shear strength capacities. As an example, the shear strength calculation for beam B48 located on Story 1 (**Figure 4.40**) is detailed below.

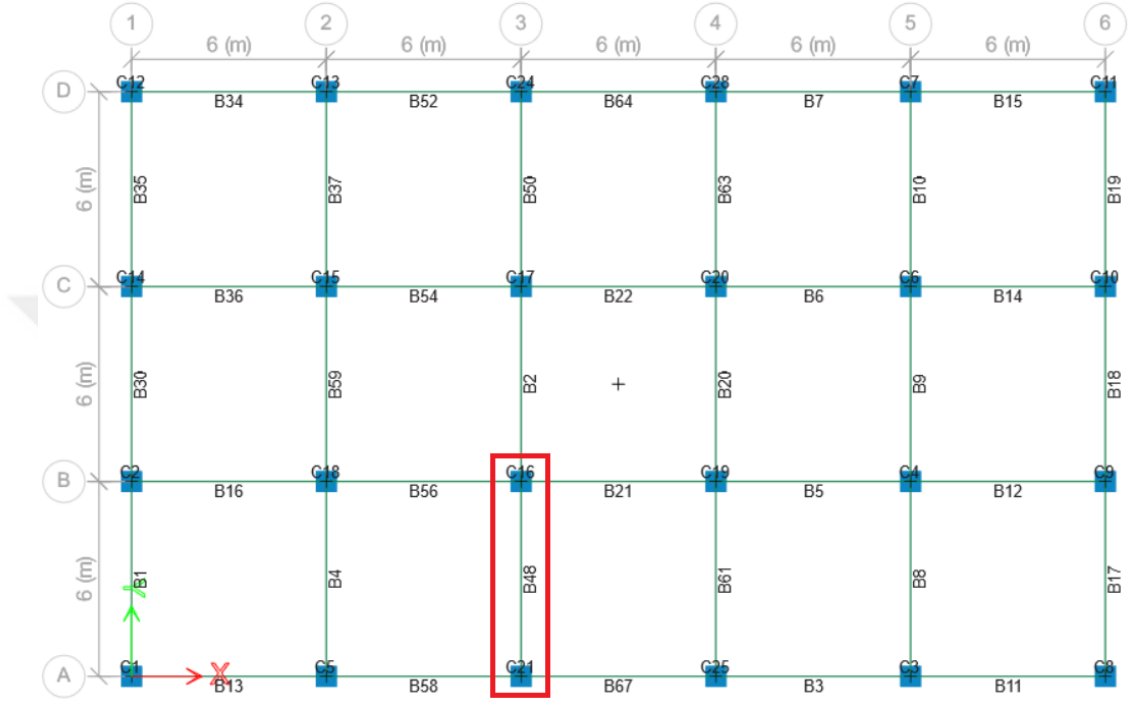


Figure 4.40: Story 1 B48 beam.

$$V_e \leq V_r \quad (4.10)$$

$$V_e \leq 0.85 * bw * d * \sqrt{f_{ck}} \quad (4.11)$$

$$bw = 400 \text{ mm}; d = 570 \text{ mm}; f_{ck} = 30 \text{ Mpa}$$

$$0.85 * 400 * 570 * \sqrt{30} = 1061.48 \text{ kN}$$

$$V_e = 582.07 \text{ kN} \leq 1061.48 \text{ kN} \quad (OK)$$

$$V_{cr} = 0.65 * f_{ctd} * bw * d * \left(1 + \gamma * \frac{Nd}{Ac}\right) \quad (4.12)$$

$$f_{ctd} = 1.28 \text{ Mpa}; Nd = 0;$$

$$Nd = 0$$

$$Ac = bw * h \quad (4.13)$$

$$bw = 400 \text{ mm}; h = 600 \text{ mm}$$

$$Ac = 400 * 600 = 240000 \text{ mm}^2$$

$$V_{cr} = 0.65 * 1.28 * 400 * 570 * (1 + 0) = 189.69 \text{ kN}$$

$$V_w = \frac{A_{sw}}{s} * f_{ywd} * d \quad (4.14)$$

$$A_{sw} = 2 * 113.04 = 226.08 \text{ mm}^2$$

$$s = 100 \text{ mm}; f_{ywd} = 365 \text{ Mpa}$$

$$V_w = \frac{226.08}{100} * 365 * 570 = 470.36 \text{ kN}$$

$$V_r = V_c + V_w \quad (4.15)$$

$$V_c = 0.8 V_{cr}$$

$$V_c = 0.8 * 189.69 \text{ kN} = 151.752 \text{ kN}$$

$$V_r = 151.752 + 470.36 = 622.11 \text{ kN}$$

$$V_r = 622.11 \text{ kN} ; V_e = 582.07 \text{ kN}$$

$$V_r > V_e \rightarrow OK$$

The **Table 4.11** presents the shear force checks corresponding to brittle failure for the beams located on the first story only.

Table 4.11: Beam Shear force check for Building A.

Story	Beam Label	V_e (kN)	V_{cr} (kN)	V_w (kN)	V_c (kN)	V_r (kN)	V_{max} (kN)	Result
Story1	B1	211.51	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B4	246.61	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B13	281.42	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B16	251.71	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B30	357.87	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B48	582.08	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B50	246.30	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B52	501.03	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B54	417.96	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B56	235.61	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B58	193.48	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B59	425.38	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B61	503.04	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B63	241.18	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B64	232.20	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B67	233.91	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B3	382.02	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B5	363.02	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B6	246.20	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B7	223.42	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B8	204.23	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B9	221.15	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B10	239.39	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B11	280.84	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B12	285.17	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B14	289.43	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B15	198.99	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B17	192.76	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B18	201.56	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B19	291.87	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B29	247.75	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B33	297.87	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B34	253.83	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B35	187.21	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B36	288.37	189.70	470.36	151.76	622.12	1061.49	OK
Story1	B37	214.03	189.70	470.36	151.76	622.12	1061.49	OK

4.2.13. Building B Beam Shear Force Check

According to the example shear force calculation performed for Building A, the maximum shear force acting on a beam element in this building is 475.23 kN in absolute value. Since all the beams in this building share the same material and cross-sectional properties as those in Building A, the shear forces in all beam elements remain within the limits specified by TBEC 2018. Therefore, all beam elements in this block are deemed adequate in terms of shear strength.

4.2.14. Building C Column Shear Force Check

According to the example shear force calculation performed for Building A, the maximum shear force acting on a column element in this building is 557.03 kN in absolute value. Since all columns in this building have the same material and cross-sectional properties as those in Building A, the shear forces in all column elements remain within the limits specified by TBEC 2018. Therefore, all column elements in this block are deemed adequate in terms of shear strength..

4.2.15. Building A Column Shear Force Check

For the evaluation of brittle failure, shear force demands were compared with shear strength capacities. As an example, the shear strength calculation for column C4 located on Story 2 (**Figure 4.41**) is detailed below.

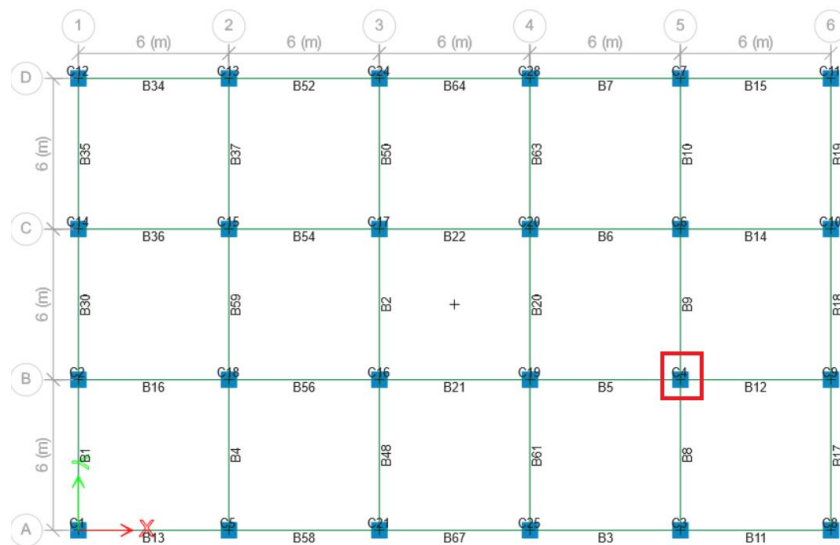


Figure 4.41: Story 2 C4 column.

$$V_e \leq V_r \quad (4.10)$$

$$V_e \leq 0.85 * bw * d * \sqrt{f_{ck}} \quad (4.11)$$

$$A_w \leq bw * d$$

$$bw = 650 \text{ mm}; d = 625 \text{ mm}; f_{ck} = 30 \text{ Mpa}$$

$$0.85 * 650 * 625 * \sqrt{30} = 1891.35 \text{ kN}$$

$$V_e = 850.22 \text{ kN} \leq 1891.35 \text{ kN} \quad (OK)$$

$$V_{cr} = 0.65 * f_{ctd} * bw * d * \left(1 + \gamma * \frac{Nd}{Ac}\right) \quad (4.12)$$

$$f_{ctd} = 1.28 \text{ Mpa}; Nd = 0;$$

$$Nd = 2355.83 \text{ kN}$$

$$Ac = bw * h \quad (4.13)$$

$$bw = 650 \text{ mm}; h = 625 \text{ mm}$$

$$Ac = 650 * 625 = 406250 \text{ mm}^2$$

$$V_{cr} = 0.65 * 1.28 * 650 * 625 * \left(1 + 0.07 * \frac{2355830}{406250}\right) = 338.14 \text{ kN}$$

$$V_w = \frac{A_{sw}}{s} * f_{ywd} * d$$

$$A_{sw} = 4 * 113.04 = 452.16 \text{ mm}^2$$

$$s = 100 \text{ mm}; f_{ywd} = 365 \text{ Mpa}$$

$$V_w = \frac{452.16}{100} * 365 * 625 = 1031.49 \text{ kN}$$

$$V_r = V_c + V_w$$

$$V_c = 0.8 V_{cr}$$

$$V_c = 0.8 * 338.14 \text{ kN} = 270.51 \text{ kN}$$

$$V_r = 270.51 + 1031.49 = 1302 \text{ kN}$$

$$V_r = 1302 \text{ kN}; V_e = 850.22 \text{ kN}$$

$$V_r > V_e \rightarrow OK$$

The **Table 4.12** presents the shear force checks corresponding to brittle failure for the columns located on the first story only.

Table 4.12: Column shear force check for Building A.

Story	Column	P (KN)	Ve (KN)	Vcr (KN)	Vw (KN)	Vc (KN)	Vr (KN)	Vmax (KN)	Result
Story1	C1	828.54	558.50	338.05	1031.49	270.44	1301.93	1891.35	OK
Story1	C2	204.87	651.53	338.01	1031.49	270.41	1301.90	1891.35	OK
Story1	C5	-544.54	765.48	338.03	1031.49	270.43	1301.92	1891.35	OK
Story1	C18	-2658.59	780.44	338.15	1031.49	270.52	1302.01	1891.35	OK
Story1	C21	-551.28	764.32	338.03	1031.49	270.43	1301.92	1891.35	OK
Story1	C24	-678.35	747.48	338.04	1031.49	270.43	1301.92	1891.35	OK
Story1	C25	-552.67	760.29	338.03	1031.49	270.43	1301.92	1891.35	OK
Story1	C28	-679.58	746.90	338.04	1031.49	270.43	1301.92	1891.35	OK
Story1	C3	-549.06	763.79	338.03	1031.49	270.43	1301.92	1891.35	OK
Story1	C4	-2662.72	776.74	338.16	1031.49	270.52	1302.01	1891.35	OK
Story1	C6	-2686.77	774.78	338.16	1031.49	270.53	1302.02	1891.35	OK
Story1	C7	-686.25	746.04	338.04	1031.49	270.43	1301.92	1891.35	OK
Story1	C8	438.38	671.18	338.03	1031.49	270.42	1301.91	1891.35	OK
Story1	C9	174.18	468.79	338.01	1031.49	270.41	1301.90	1891.35	OK
Story1	C10	170.82	469.33	338.01	1031.49	270.41	1301.90	1891.35	OK
Story1	C11	818.49	644.03	338.05	1031.49	270.44	1301.93	1891.35	OK
Story1	C12	631.75	449.96	338.04	1031.49	270.43	1301.92	1891.35	OK
Story1	C13	-633.26	747.97	338.04	1031.49	270.43	1301.92	1891.35	OK
Story1	C14	190.16	533.47	338.01	1031.49	270.41	1301.90	1891.35	OK
Story1	C15	-2689.42	777.53	338.16	1031.49	270.53	1302.02	1891.35	OK
Story1	C16	-2660.64	782.55	338.15	1031.49	270.52	1302.01	1891.35	OK
Story1	C17	-2688.69	774.96	338.16	1031.49	270.53	1302.02	1891.35	OK
Story1	C19	-2647.86	785.80	338.15	1031.49	270.52	1302.01	1891.35	OK
Story1	C20	-2684.20	777.71	338.16	1031.49	270.53	1302.02	1891.35	OK

4.2.16. Building B & Building C Shear Wall Shear Force Check

For the evaluation of brittle failure, shear force demands were compared with shear strength capacities. As an example, the shear strength calculation for wall V22, labeled as P02 on Story 1 (**Figure 4.42**), is detailed below.

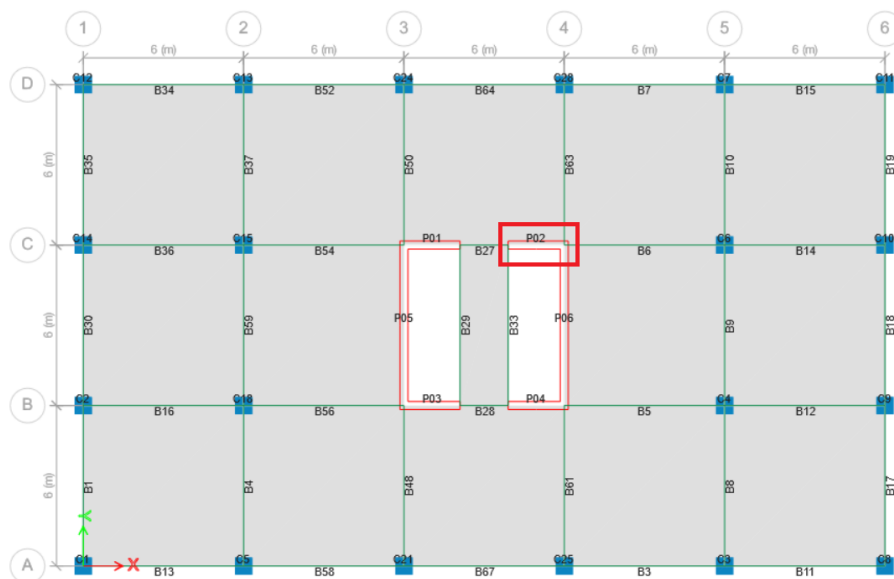


Figure 4.42: Story 1 P02 shear wall.

$$V_e \leq V_r$$

$$V_e \leq 0.85 * A_{ch} * \sqrt{f_{ck}}$$

$$A_{ch} \leq b_w * h$$

$$b_w = 300 \text{ mm}; h = 2100 \text{ mm}; f_{ck} = 30 \text{ Mpa}$$

$$0.85 * 300 * 2100 * \sqrt{30} = 2933.05 \text{ kN}$$

$$V_e = 4032.88 \text{ kN} \geq 2933.05 \text{ kN} \text{ (not ok)}$$

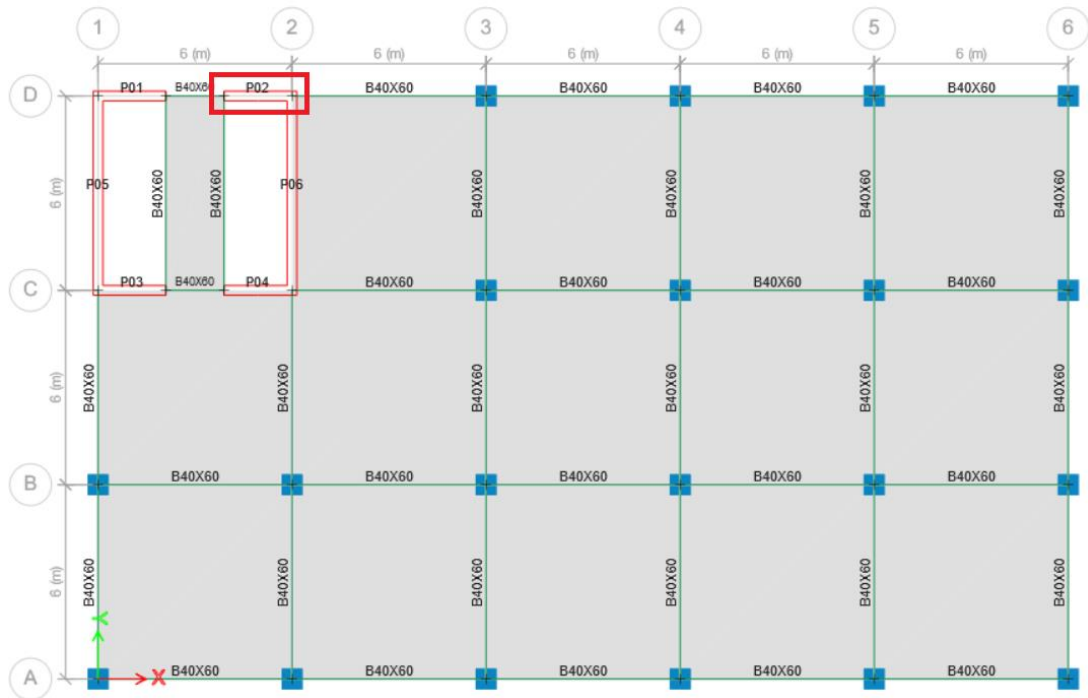


Figure 4.43: Story 1 P02 beam.

$$V_e \leq V_r$$

$$V_e \leq 0.85 * A_{ch} * \sqrt{f_{ck}}$$

$$A_{ch} \leq b_w * h$$

$$b_w = 300 \text{ mm}; h = 2100 \text{ mm}; f_{ck} = 30 \text{ Mpa}$$

$$0.85 * 300 * 2100 * \sqrt{30} = 2933.05 \text{ kN}$$

$$V_e = 4726.12 \text{ kN} \geq 2933.05 \text{ kN} \text{ (not ok)}$$

The **Table 4.13** presents the shear force checks corresponding to brittle failure for the shear walls.

Table 4.13: Shear wall shear force check for Building B and C.

Building B Pier Forces						
Story	Pier	V22 (kN)	b (cm)	h (cm)	Vmax (kN)	Result
Story6	P01	1035.13	30	210	2933.05	OK
Story6	P02	1115.59	30	210	2933.05	OK
Story6	P03	1052.86	30	210	2933.05	OK
Story6	P04	1153.49	30	210	2933.05	OK
Story6	P05	1364.72	30	600	8380.16	OK
Story6	P06	1500.11	30	600	8380.16	OK
Story5	P01	1206.81	30	210	2933.05	OK
Story5	P02	1572.51	30	210	2933.05	OK
Story5	P03	1426.68	30	210	2933.05	OK
Story5	P04	1513.81	30	210	2933.05	OK
Story5	P05	3872.27	30	600	8380.16	OK
Story5	P06	3496.38	30	600	8380.16	OK
Story4	P01	1165.61	30	210	2933.05	OK
Story4	P02	1928.97	30	210	2933.05	OK
Story4	P03	1798.67	30	210	2933.05	OK
Story4	P04	1610.55	30	210	2933.05	OK
Story4	P05	5611.37	30	600	8380.16	OK
Story4	P06	4691.75	30	600	8380.16	OK
Story3	P01	1722.08	30	210	2933.05	OK
Story3	P02	2234.78	30	210	2933.05	OK
Story3	P03	1857.40	30	210	2933.05	OK
Story3	P04	1974.70	30	210	2933.05	OK
Story3	P05	6311.48	30	600	8380.16	OK
Story3	P06	4710.77	30	600	8380.16	OK
Story2	P01	2140.34	30	210	2933.05	OK
Story2	P02	2844.87	30	210	2933.05	OK
Story2	P03	2522.24	30	210	2933.05	OK
Story2	P04	2506.59	30	210	2933.05	OK
Story2	P05	6954.64	30	600	8380.16	OK
Story2	P06	5565.14	30	600	8380.16	OK
Story1	P01	3246.31	30	210	2933.05	NOT OK
Story1	P02	4032.88	30	210	2933.05	NOT OK
Story1	P03	3627.04	30	210	2933.05	NOT OK
Story1	P04	3578.11	30	210	2933.05	NOT OK
Story1	P05	8725.59	30	600	8380.16	NOT OK
Story1	P06	6920.76	30	600	8380.16	OK

Building C Pier Forces						
Story	Pier	V22 (kN)	b (cm)	h (cm)	Vmax (kN)	Result
Story6	P01	938.42	30	210	2933.05	OK
Story6	P02	968.68	30	210	2933.05	OK
Story6	P03	1322.94	30	210	2933.05	OK
Story6	P04	1503.04	30	210	2933.05	OK
Story6	P05	1638.38	30	600	8380.16	OK
Story6	P06	2827.50	30	600	8380.16	OK
Story5	P01	1679.72	30	210	2933.05	OK
Story5	P02	1559.58	30	210	2933.05	OK
Story5	P03	1135.99	30	210	2933.05	OK
Story5	P04	1431.87	30	210	2933.05	OK
Story5	P05	2722.87	30	600	8380.16	OK
Story5	P06	3033.16	30	600	8380.16	OK
Story4	P01	1548.14	30	210	2933.05	OK
Story4	P02	1749.27	30	210	2933.05	OK
Story4	P03	1779.48	30	210	2933.05	OK
Story4	P04	1829.98	30	210	2933.05	OK
Story4	P05	3655.48	30	600	8380.16	OK
Story4	P06	3934.60	30	600	8380.16	OK
Story3	P01	2065.58	30	210	2933.05	OK
Story3	P02	2221.65	30	210	2933.05	OK
Story3	P03	2125.86	30	210	2933.05	OK
Story3	P04	2208.86	30	210	2933.05	OK
Story3	P05	4982.35	30	600	8380.16	OK
Story3	P06	5278.20	30	600	8380.16	OK
Story2	P01	1819.89	30	210	2933.05	OK
Story2	P02	3217.38	30	210	2933.05	OK
Story2	P03	2269.38	30	210	2933.05	OK
Story2	P04	2544.53	30	210	2933.05	OK
Story2	P05	5990.44	30	600	8380.16	OK
Story2	P06	7038.36	30	600	8380.16	OK
Story1	P01	3093.37	30	210	2933.05	NOT OK
Story1	P02	4726.12	30	210	2933.05	NOT OK
Story1	P03	4213.95	30	210	2933.05	NOT OK
Story1	P04	4783.35	30	210	2933.05	NOT OK
Story1	P05	7525.10	30	600	8380.16	NOT OK
Story1	P06	8876.81	30	600	8380.16	OK

4.2.17. Energy Calculations Result

The energy outputs from the analyses conducted for different building types are examined in this section. **Figure 4.44**, **Figure 4.45** and **Figure 4.46** shows total energy components results.

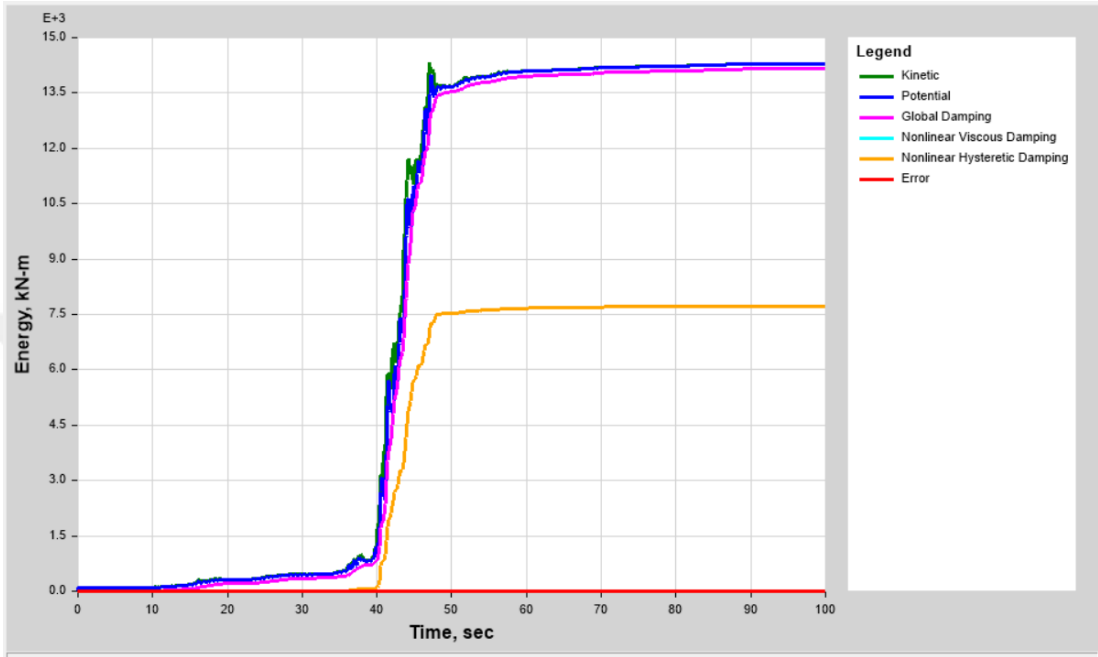


Figure 4.44: Building A cumulative energy components results.

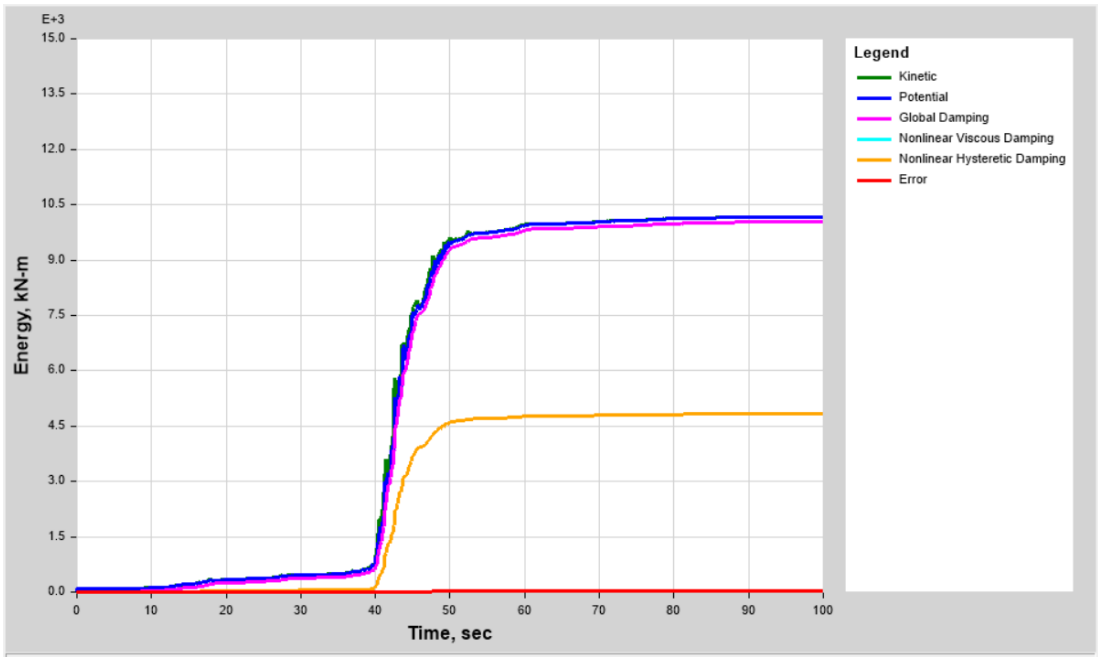


Figure 4.45: Building B cumulative energy components results.

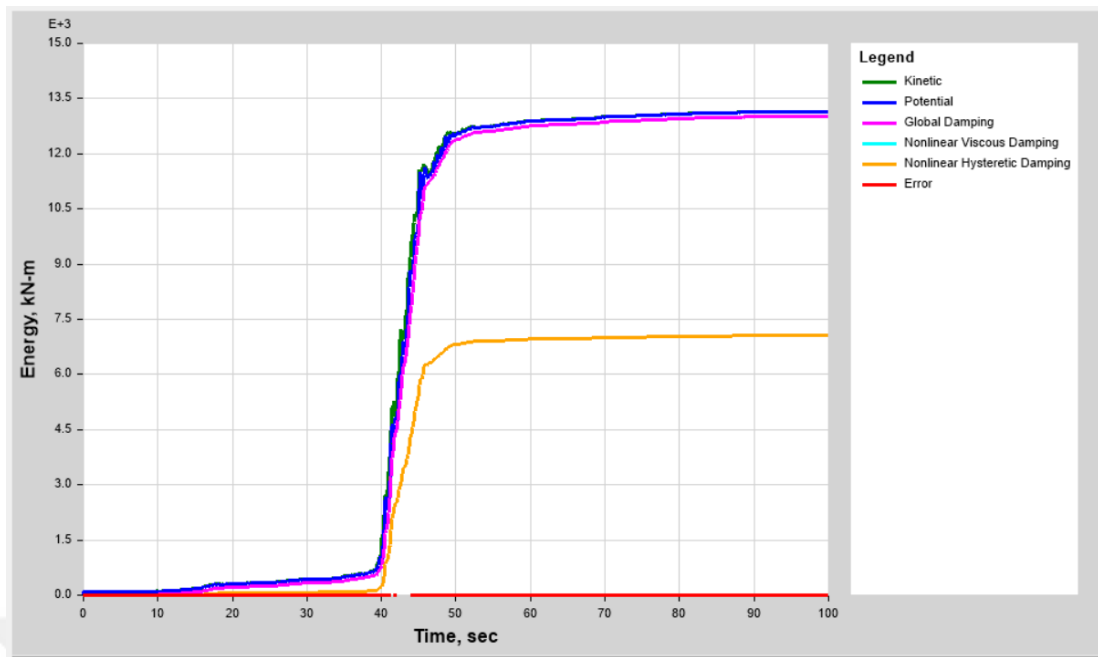


Figure 4.46: Building C cumulative energy components results.

Figure 4.47, Figure 4.48 and Figure 4.49 shows hysteretic damping energy for group members.

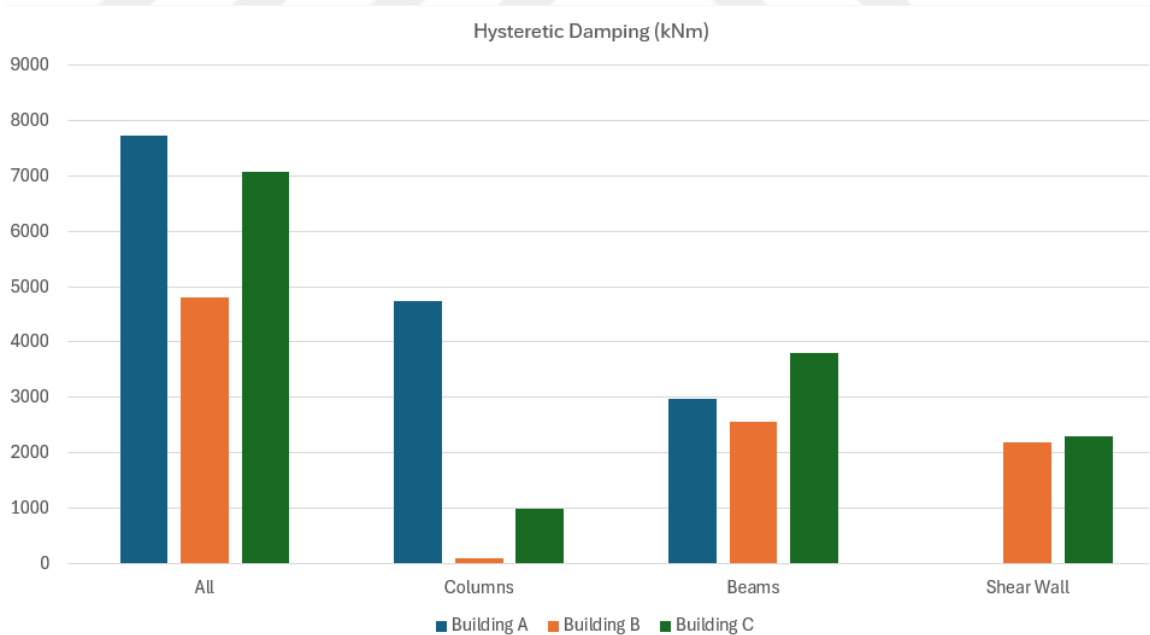


Figure 4.47: Hysteretic damping energy values for group members.

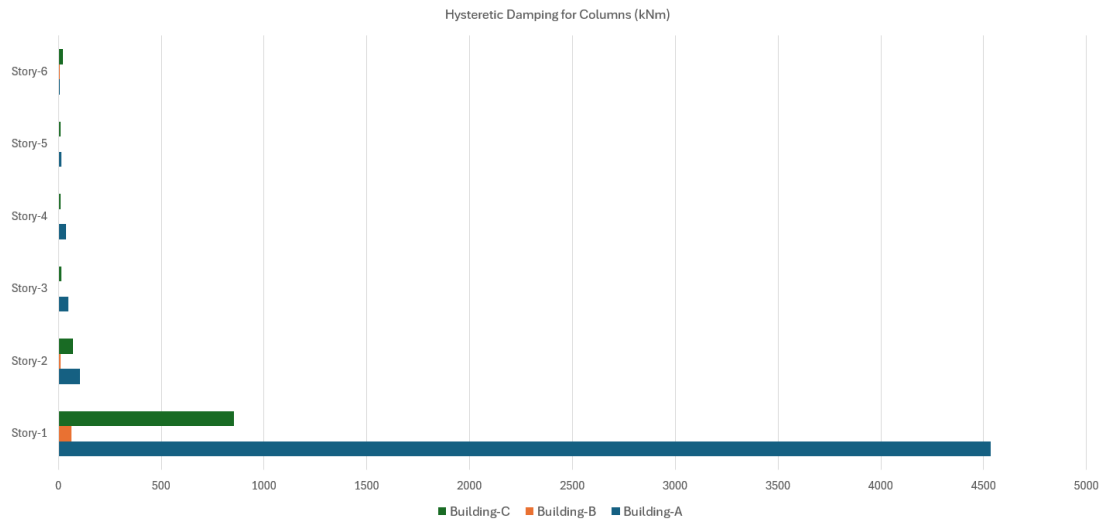


Figure 4.48: Hysteretic damping energy values for columns.

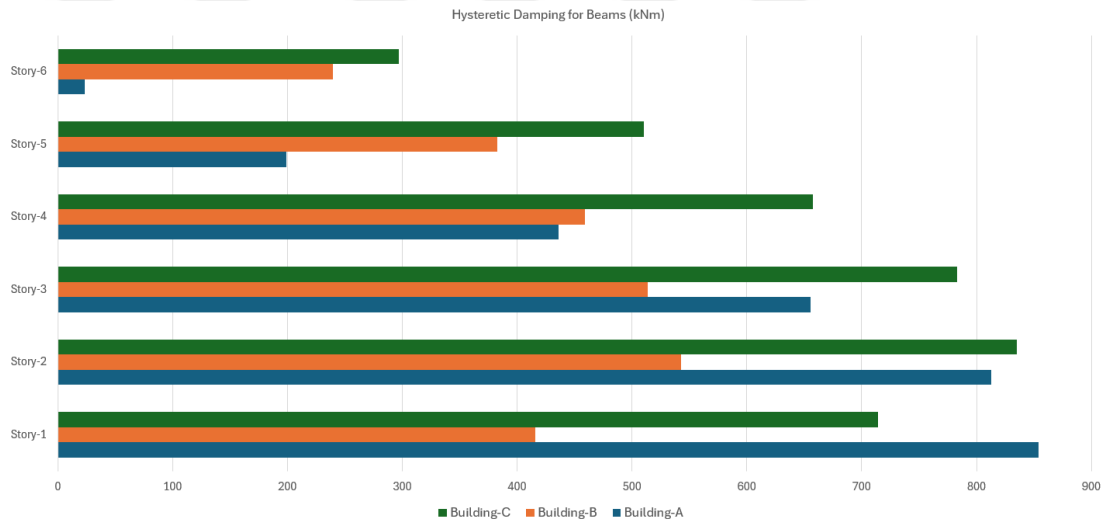


Figure 4.49: Hysteretic damping energy values for beams.

5. CONCLUSION

Within the scope of this thesis, three buildings representing three different structural systems were analyzed using the Nonlinear Time History Analysis method in accordance with the 2018 Turkish Building Earthquake Code (TBEC). The first structure is a reinforced concrete frame system without torsional irregularity. This structure is referred to as Building A in this study. The second structure is a shear wall-frame system without torsional irregularity. The third structure is also a shear wall-frame system; however, it includes torsional irregularity. These two structures are referred to as Building B and Building C. For the analyses, earthquake records from the magnitude 7.8 earthquake that occurred on February 6, 2023 in the Pazarcık district of Kahramanmaraş Province were used. For all three buildings, data from the AFAD station TK-2718 located in the Islahiye district of Gaziantep Province were utilized. It was assumed that all three buildings are located at the coordinates of the recording station, with a local soil class of ZC, and the earthquake records were used without scaling.

As required by all modern seismic codes, a structural system must possess adequate strength, stiffness, and ductility. If a structure fails to meet one or more of these criteria, it may not achieve the intended performance level for its structural elements or the system as a whole when subjected to a strong earthquake. This can increase the likelihood of damage and may even lead to total structural collapse. To prevent such outcomes, both the structure and its components must be designed to ensure adequate strength, stiffness, and ductility.

Building A, which consists of high-ductility frames where all seismic loads are resisted by frames only and without any shear walls, has lower lateral stiffness compared to the other buildings. As a result, this structure is expected to experience greater displacements, and the plastic deformation zones of its seismic load-resisting elements will undergo more inelastic deformations than those in other systems. This indicates a higher ductility demand for this system. However, due to its higher degree of redundancy compared to the other systems, the number of points at which earthquake energy can be dissipated is also greater. If designed to avoid second-order effects and to prevent brittle failure in structural elements, such systems can be defined as the most

effective conventional reinforced concrete systems for dissipating seismic energy. In this thesis study, no second-order effects were observed in Building A, and no brittle failure was allowed in its structural elements. Therefore, Building A is considered to have adequate strength, stiffness, and ductility under nonlinear analysis in accordance with TBEC 2018.

Building B, in which all seismic loads are resisted by high-ductility shear walls in combination with high-ductility frames, features a symmetrical wall layout in both principal directions. Since this structure includes shear walls, it possesses high lateral stiffness. Therefore, it undergoes smaller displacements compared to the other structures, and the inelastic deformation in the plastic hinge zones of its seismic load-resisting frame elements is minimized. For the wall elements, the strain values corresponding to ductile behavior remain below the limits defined in TBEC 2018. Thus, the ductility demand of this system is lower than that of Building A. However, due to the high stiffness contribution of the walls relative to the overall story stiffness, most of the story shear forces are resisted by the walls. According to the analysis results, shear forces in the first-story walls exceed the allowable limits defined by the code. This means that although no second-order effects occur and strain demands remain within acceptable ductile behavior limits, the force demands corresponding to brittle behavior exceed the allowable thresholds. The takeaway from this finding is that, when designing such systems, the added elements (e.g., walls) must have adequate strength. Therefore, Building B, under nonlinear analysis in accordance with TBEC 2018, does not have adequate strength, although it does have sufficient stiffness and ductility.

Building C, in which all seismic loads are resisted by high-ductility shear walls in combination with high-ductility frames, is characterized by torsional irregularity. Although this structure includes shear walls and therefore exhibits high lateral stiffness, due to the torsional irregularity, the system is subjected to both translational and rotational displacements. As a result, higher lateral displacements are expected to occur at the corner points of the building façade where there are no shear walls, compared to Building B. Compared to Building A, this structure undergoes less lateral displacement. The amount of inelastic deformation in the plastic hinge regions of the frame elements that transfer seismic loads remains at a level between the other two alternatives. For the wall elements, the strain values of the concrete and reinforcement

fibers corresponding to ductile behavior remain below the limits specified in TBDY 2018. The ductility demand of this system is lower than that of Building A. However, since the stiffness of the walls constitutes a very high proportion of the story stiffness, a large portion of the story shear forces is resisted by the walls. Due to the presence of torsional irregularity in the system, plastic deformation values in the frame elements distant from the walls are close to those in Building A, whereas in frames near the walls, the opposite is observed. Although second-order effects do not occur in Building C, and the deformation limits corresponding to ductile behavior are not exceeded, the strength limits corresponding to brittle behavior are exceeded. This highlights the necessity of ensuring that the structural elements added to such systems possess adequate strength. Therefore, under nonlinear analysis based on TBDY 2018 provisions, Building C does not have sufficient strength, although it does possess adequate stiffness and ductility.

At the end of the study, the energy dissipation values of the structures were examined. Although energy-based design is not currently used in practice, it can be a useful tool for evaluating the global behavior and the extent to which structural elements have entered the inelastic region. The analysis results showed that Building A had the highest hysteretic damping value. Additionally, it was observed that Building C had a higher hysteretic damping value than Building B.

As a result of this study, it is clear that the simplicity and clarity of the selected structural system in any building significantly contribute to the structural behavior aligning with the expectations defined by design codes. A structural system layout that disrupts stiffness balance or causes additional internal forces in structural members due to torsional effects requires accurate determination of the strength demands associated with brittle failure modes. Otherwise, despite establishing a structurally stiffer system, elements with insufficient strength may be incorporated, leading to an overall inadequate structural capacity. In project development phases, care must be taken to avoid placing shear walls that lack adequate strength solely to conform to architectural layouts or that may induce undesirable torsional behavior in the system. Furthermore, since TBDY 2018 does not define an upper limit for torsional irregularity, special attention must be given to the placement and design of structural system elements in buildings with torsional irregularities.

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BIOGRAPHY

Alparslan DURMUŞ graduated from Pendik Anatolian High School in 2003. He graduated from Istanbul Technical University with a degree in Civil Engineering in 2010. He worked for 3 years at a structural design office starting from 2011. Afterwards, he established his own company providing engineering services. In 2022, he started his master's degree in Earthquake and Structural Engineering at Gebze Technical University.



PUBLICATIONS AND PRESENTATIONS FROM THE THESIS

Durmuş, A., Mert, Ü. (2024). Investigation of The Performance of Different Types of Reinforced Concrete Structures Utilizing Nonlinear Time History Analysis. Sustainable Resilient Future. 2024 6th International Resilience Congress

